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Water Purification Division

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WATER PURIFICATION PROBLEMS IN MINING AND MANUFACTURING DISTRICTS

By C. F. Drake[1]

Within the memory of many of the members of the American Water Works Association, the fitness of water for municipal supply was determined almost entirely by chemical analysis, supplemented by a careful examination of the watershed.

With the adoption of improved bacteriological methods in water analysis, however, chemical analysis became of somewhat less moment. Now that the furnishing of a bacterially safe drinking water is being very generally accomplished in nearly all American cities, and in a large part of rural America, it appears that communities are again becoming more exacting concerning the physical and chemical character of public water supplies.

On October 21, 1914, the well-known "Treasury Department Standard for the Examination of Water of Interstate Common Carriers" was made public. In May, 1922, the Surgeon General appointed a committee to review the Treasury Department standard for drinking water on interstate common carriers, and to recommend a standard or standards, based on specific methods of laboratory analysis and field surveys, to be applicable to all classes of water supplies coming within the jurisdiction of the interstate quarantine regulations of the United States.

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"Public Health Reports," April 10, 1925, sets forth the report of this Advisory Committee.   A portion of the report is as follows:

"Appendix IV.   The physical and chemical characteristics of acceptable water supplies.   Definition of a satisfactory water.

With respect to chemical and physical characteristics a water, to be suitable for drinking and culinary purposes on common carriers engaged in interstate traffic, should be clear, colorless, odorless, pleasant to the taste, should be free from toxic salts, and should not contain an excessive amount of soluble mineral substances, or of any chemicals employed in treatment.

Turbidity should not exceed 10 (Silica scale) and in general it should not be more than 5.

Lead shall not exceed 0.1 p.p.m.

Copper shall not exceed 0.2 p.p.m.

Sulfate (SO_4) should not exceed 250 p.p.m.

Magnesium should not exceed 100 p.p.m.

Total solids should not exceed 1000 p.p.m.

Chlorides should not exceed 250 p.p.m.

Iron should not exceed 0.3 p.p.m.

The water should contain no caustic alkalinity.

The water should have no odor or taste of free chlorine.

The water should contain a residual alkalinity of at least 10 p.p.m., if it has been treated with sulfate of aluminum or other aluminum compound.

The carbonates of sodium and potassium, taken together and calculated as normal calcium carbonate, should not exceed 50 p.p.m."

In general, there is little difficulty in approaching these standards without excessive cost or ingenuity.   But many communities impose added and more exacting requirements upon the water supplied. Unless there is further popular education, especially along economic lines, concerning the effect produced by such refinements as will ultimately be demanded by many communities, operators of water purification plants will be very unpopular members of society.

The Allegheny River and the Monongahela River are the sources of water supply of Pittsburgh and of many other cities and boroughs in this vicinity.

With extensive coal mining and industrial establishments throughout the district, it is not surprising that the streams are the recipients of wastes of very varied, sometimes offensive, character.

On the Allegheny River, are such industries as:

Steel Mills	Wood Alcohol Plants
Coal Mining	Paper Pulp Mills
Oil Production and Refinery	Glass Factories
Natural Gas	Brick Yards

Tanneries   Aluminum Ware Factories
Coke Ovens   Etching Plants
Lumber Camps   Glue Factories

One of the latest industries on the Allegheny River, about 15 miles above the intake for the Pittsburgh supply is the largest chromium steel plant in the country.   The quantity and character of discharge is not yet known.   With this wide diversity of industries, and with the resulting wide diversity of industrial wastes, it is not surprising that the problem of producing a municipal water supply meeting the increasingly exacting chemical and physical requirements of the consumer is difficult and expensive.   But the difficulty is, in general,

TABLE 1

Pounds, per million gallons, of various chemicals required to neutralize various parts per million acidity

ACIDITY	FREE AND HALF-BOUND CO_2	METHYL RED ACIDITY, EQUIVALENT $CaCO_3$		
	85 percent CaO	85 percent CaO	95 percent CaO_2H_2	97 percent Na_2CO_3
p.p.m.				
1	12.5	5.5	6.5	9.1
2	25.0	11.0	13.0	18.2
3	37.5	16.5	19.5	27.3
4	50.0	22.0	26.0	36.4
5	62.5	27.5	32.5	45.5
6	75.0	33.0	39.0	54.6
7	87.5	38.5	45.5	63.7
8	100.0	44.0	52.0	72.8
9	112.5	49.5	58.5	81.9
10	125.0	55.0	65.0	91.0

along the line of economics rather than along the line of chemistry or bacteriology.

Due to mine and industrial wastes, the annual average alkalinity of the Allegheny River at Aspinwall has fallen from 24, in 1909, to 5 p.p.m. in 1929.   During the same period, the total soap hardness of the river water at Aspinwall rose from 67.9 to 76.9 p.p.m.   Analyses have shown quite conclusively that the decreased alkalinity has been brought about by wastes from and below the Kiskiminetas River.   The Kiskiminetas is approximately 22 miles above the Pittsburgh intake at Aspinwall.

The Kiskiminetas River receives the untreated drainage from a

large number of coal mines. It also receives the acid pickling liquid from many large steel mills, including those of the Johnstown district.

In 1929, the waterworks at Aspinwall used 358,650 pounds of soda ash, costing $6,961.18, to produce an applied water neutral to methyl red, during the periods of acid river water. Had we built up a substantial alkalinity throughout the year, the cost of soda ash would have been many times greater. At times, soda ash was applied at the rate of 1200 pounds per hour. Application was made at the rate of 0.4 pound per hour of 58 percent soda ash per part per million acidity, for every million gallons of water treated per day. In this connection, table 1 shows the quantities of lime or soda ash required to neutralize acid water.

We used soda ash, rather than lime and soda ash treatment, because we had no facilities for handling excessive quantities of sludge. It has frequently been urged that Pittsburgh should build a plant to treat more completely the acid river waters which occur every summer, for varying periods generally of about three months' duration.

As we have been assured that maximum intensity of mine waste discharge has already been reached, and that there is reasonable ground for the assumption that improved conditions may be anticipated, however, it is probable that a large expenditure for a plant is not warranted. Beside the increased hardness and acidity caused by mine waste, it has been found that the presence of manganese is marked. In many communities, the manganese in the water causes the offensive brown stains in the laundry.

The question sometimes arises as to difficulties in purifying waters with acid mine drainage. In the first place, the rapid and wide fluctuation in acid content is of much consequence. A constant load might readily be cared for, but the uncertainty results in poor flocs for rapid sand filters and sterilized beds for slow sand filters.

It has frequently been stated that acid waste results in partial or complete sterilization of the water to which it is applied. If the treatment were constant, there would be weight to the statement. If, however, conditions are such that there are unexpected changes from acid to alkaline, acidity sterilization causes a false sense of security. If the method of purification is not efficient without the assistance of indefinite acid action, the method of purification should be improved.

The unreliability of mine waste sterilization is brought about by the fact that the pumpage from the mines is usually scheduled to gain the advantage of off-peak prices for the electric current used to operate the pumps.   As a result, only the most extraordinary good fortune will produce an even distribution of the acid waste throughout the day.

We at Aspinwall have noted a cycle of seven days in alkalinity fluctuation.   This is due to the work schedule of the pump operator. It has been stated that the discharge from the pickling vats of steel mills has a very considerable influence upon the acidity of streams. An exhaustive study of that feature has been made and is being made by the Pennsylvania Sanitary Water Board.   Until they have reported their findings, it might be well to withhold judgment.

In line with coal mine problems, is the problem of coking.   The Allegheny River has had but little trouble with phenol tastes.   The Monongahela River, however, has had considerable trouble in the past.   It should be said that those interested in by-product coke plants have devoted a full measure of thought and care to the prevention of phenol presence in all waters upon which reliance is placed for water supply.

The situation in Western Pennsylvania may be summarized as follows:

1. Water purification problems in that area are in line with those of other areas with large and diversified industrial developments.

2. Vital statistics indicate that inhabitants of that area are being as well protected from water-borne disease as are the inhabitants of other sections of the country.

3. The regulation of the introduction of objectionable wastes into the streams is being given careful study by representatives of the States, who are constantly protecting public health.

Beyond that, they are constantly balancing the economic gain in improving the physical condition of the streams against the economic loss to basic industries in bringing about such improvements.

Probably no single large portion of the country has water problems which must be handled more delicately.

The most encouraging aspect of the situation is the fact that the industries are lending themselves to the solution of the problems, as far as financial limitations brought about by sectional competition for industrial supremacy will permit.

DISCUSSION

EDWARD C. TRAX:[2] This paper by Mr. Drake and his companion paper, "Effect of Acid Mine Drainage on River Water Supply," read in the main session this morning, are a very complete exposition of the stream pollution problems in this district and will be invaluable to any one who has occasion to delve into this subject in the future.

In the Monongahela valley this pollution has reached a very serious stage. The Monongahela River and its tributaries drain what is probably the most actively developed bituminous coal field in the country. With very few exceptions no attempts have been made to treat or neutralize the acid and acid salts in the mine drainage and it has been almost universal practice to discharge these wastes into the most convenient nearby water course. Many large manufacturing plants are located on the watershed, most of them connected with the iron and steel industry. Various other industries are represented, coke ovens are numerous, and there are two by-product coke plants. Phenolic wastes from these plants have given much trouble in the past, but they are now almost perfectly controlled and seldom reach the river in troublesome amounts.

By far the most serious problem we have to contend with is the acid-iron wastes from coal mines. The composition of mine drainage varies widely. Alkaline mine waters are unusual, but by no means rare. Usually the drainage contains large amounts of the sulphates of iron, aluminum, calcium, and magnesium, and free sulphuric acid. The presence of a small amount of acidity is not objectionable in the purification of the water for domestic purposes. Sufficient sulphates of iron and aluminum to thoroughly coagulate the water are present in the river practically all of the time. The usual procedure for coagulation is therefore reversed, it being necessary to add an alkali such as lime, soda ash, or caustic soda, to neutralize the acid and precipitate the iron and aluminum. These acid waters when properly treated and filtered come out beautifully clear and bright and are very palatable. As Mr. Drake has pointed out, it is the extreme variability of the water and peak conditions which make the problem most difficult. Variations in hardness of 300 p.p.m. within three hours have been observed in the Youghiogheny River at McKeesport. The discharge of drainage from the mines is fairly constant in quantity and the concentration of acid in the streams is largely influenced

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by conditions of rainfall and runoff, particularly by distribution of rainfall. It is during periods of low flows that the concentration of the mine drainage becomes particularly troublesome. This pollution has resulted in a tremendous burden of cost to the municipalities and industries using the water. This includes increased cost for construction of plants to purify and soften the water during peak load conditions, cost for chemicals and the handling of large amounts of chemicals, corrosion of raw water pumps and piping, cost for soap and domestic softening compounds in the communities using the water, etc.

The natural water over the Monongahela drainage area is generally soft and low in alkalinity and will neutralize a comparatively small amount of acid mine water. For this reason the river, and more particularly some of its large tributaries, have been acid for many years. From the meager records and information available it seems probable that the Youghiogheny River at times of low flow contained free acid in sufficient quantity to kill fish as early as 1890, and it also seems probable that the Monongahela at Pittsburgh contained free acid at times prior to 1900. From this time on the pollution has been steadily increasing. As has been stated, the quantity of drainage increases in almost direct proportion to the amount of coal mined out and the acidity and mineral content of the water from individual mines almost invariably increase as more acreage is mined out. During the past ten years the pollution of the Monongahela River has increased rapidly and we are approaching the point where, at times of low flow, with high concentration of the acid wastes, a satisfactory water for domestic purposes cannot be turned out, even from the most modern plants and with tremendous expenditure for chemicals.

It has been customary to consider the acid-iron pollution of the Monongahela River as due to wastes from coal mines and manufacturing plants connected with the iron and steel industry, and the proportion of the wastes contributed by each industry has long been a matter of controversy. In so far as acid is concerned the condition of the river is entirely due to drainage from coal mines. While it is true that some spent pickling liquor is discharged into the river from steel plants, other features in connection with the operation of these plants more than counter balance and acid wastes discharged from them. Several hundred millions of gallons of water per day are taken from the river by the steel plants for cooling purposes. It has become the universal practice to neutralize this water with lime to prevent

serious corrosion troubles. This neutral water is returned to the
river and effects a material reduction in the acid content of the water.
During the recent drouth, with very low flow in the river, this re-
sulted in a progressive reduction of the free acid in the lower forty
miles of the river from a maximum of 130 p.p.m. at Charleroi to
complete neutralization at Pittsburgh. For example, at Charleroi,
40 miles above Pittsburgh, the free acid during a certain period
ranged from 75 to 130 p.p.m., at Elizabeth, 22 miles from Pittsburgh,
40 to 70 p.p.m., at McKeesport, 15 miles from Pittsburgh, 20 to
30 p.p.m., while at Pittsburgh it was neutral or slightly alkaline.
The discharge of spent pickling liquor naturally adds to the total
sulphate content of the water, which is the real measure of this
pollution. However, it is my opinion, based upon many years ex-
perience, that in so far as the Monongahela River is concerned the
effect of this is almost negligible.

While the responsibility for the present, and future, condition of the
Monongahela River must be assumed by the coal operators, it must
be borne in mind that the prosperity and well being of this district
depends largely upon the prosperity of the coal industry. Coal
mining must, and will, go on, and even in this district which has
suffered so much from the pollution there is little sentiment in favor of
drastic legislation or any measures which will hamper or obstruct
the profitable working of the coal fields. There is apparently no
simple solution of the problem. Treatment of the water at the mine
is expensive and unsatisfactory and discriminates against the operator
who has a mine with an unusual amount of highly acid drainage.
The plan offering the most encouragement is that evolved and being
put into practice by the Sanitary Water Board of Pennsylvania with
the coöperation of a large majority of the coal operators. This will
doubtless be explained in detail by Mr. Moses.

H. E. Moses:[3] Mr. Drake in his usual clear style has presented
some of the difficulties confronting the waterworks operators in an
industrial region. He speaks from personal knowledge of the trouble
encountered in the western part of Pennsylvania.

Other sections of the State are likewise confronted with similar
and still other problems arising from the discharge of many kinds of
industrial wastes into streams. For example, waterworks located on

[3] Assistant Chief Engineer, State Department of Health, Harrisburg, Pa.

streams draining the anthracite district have the acid condition referred to plus the considerable task of keeping their intakes free from the culm produced in immense quantities in the preparation of hard coal and carried progressively downstream. This applies to eastern Pennsylvania and affects the Susquehanna, Lehigh and Schuylkill Rivers and their many tributaries.

To balance properly the interests of the public waterworks against those of the industry discharging objectionable wastes into water supply streams is a matter requiring good judgment on the part of the authorities of any industrial State who have jurisdiction over stream pollution.

This situation confronted the Sanitary Water Board of Pennsylvania upon its creation in 1923. This Board is charged with the enforcement of laws relating to the pollution of State waters and with the investigation and report upon ways and means of preventing and eliminating such pollution. In regard to sewage, the Board has ample powers, but its powers are circumscribed as regards industrial wastes.

Nevertheless, the Board, from the outset, has planned comprehensively to protect the now clean streams and to improve those already polluted. The magnitude of this task may be recognized when it is considered that Pennsylvania has an estimated total of 100,000 miles of streams, divided by estimate into 25,000 miles relatively clean streams, 40,000 miles more or less polluted and the remaining 35,000 miles ranging in pollution from those susceptible of improvement to the streams grossly contaminated. A very large part of this latter mileage is polluted by coal mine drainage from the bituminous and anthracite coal fields.

At the beginning, it looked like an almost hopeless task for there are many industrial wastes for which there are now no known, reasonable or practicable methods of treatment. Two courses were possible, one to use the limited legal power of the Board in dealing with individual plants which in the end accomplishes little, or the other course followed by the Board of gathering the different members of the industry together as a group under an agreement and in initiating steps to solve the problem of properly disposing of the industry's wastes. Such agreements have been effected with the tanneries, by-product coke plants, manufactured gas plants, pulp and paper industry and with the bituminous coal operators.

The details of this work are available in reports of the Sanitary

Water Board and it is believed that real accomplishments have been made. After several years of full scale experimentation, reasonable and practicable processes have been discovered for the treatment and disposal of waste waters resulting from the vegetable tanning of leather. The tannery companies of Pennsylvania are under agreement, after a study of their individual needs, to begin the installation of these processes.

The agreement with all of the by-product coke plants in Pennsylvania has brought about the installation of dephenolizing apparatus and other protective devices. Part of the agreement provides for the immediate reporting of accidental spills to the Department's District Engineers who in turn notify the downstream waterworks and likewise send word to the Departments of Health of States along the Ohio River below Pennsylvania. A similar agreement applies to the manufactured gas plants.

Probably the most serious industrial pollution of Pennsylvania streams is due to the drainage from coal mines whose effect in the bituminous district has been well described by Mr. Drake and Mr. Trax. The difficulties surrounding this problem seemed almost insurmountable, but, here too, real progress has been made. The writer has frequently heard the statement that mining is a basic industry and must not be disturbed, but that a new consciousness of the obligation of this industry to public water supplies has arisen is demonstrated by the attitude of the bituminous coal mine operators in making this agreement with the Sanitary Water Board and in their sincere endeavors to carry out improvement measures.

There is no known practicable method for treating mine drainage, but considerable advance has already been made by putting into effect certain things which can be done in any mining district. There are several instances of where the drainage from mine workings previously discharged through many outlets has been concentrated to a single point of discharge with the consequent redemption of many miles of stream. In one case, this also brought about the abandonment of sixty individual pumping stations. A similar result is secured by the judicious diversion of mine drainage from one watershed to another.

Impoundage of acid mine water underground with the consequent exclusion of oxygen from the acid forming materials occurs either naturally or has been brought about artificially. In quite a number of instances, the outflow by actual tests has been found alkaline.

The process gives promise of much usefulness. It is at once obvious that where this can be accomplished, the immediate saving in pumping and piping a non-corrosive water will be appreciable. This cannot be considered a panacea for all acid mine drainage troubles, but it appeals sufficiently to the coal operators to cause some of them to spend considerable money in adapting their mines to the process. Its further progress will be watched with interest.

MANGANESE IN WATER, ITS OCCURRENCE AND REMOVAL

By Robert Spurr Weston[1]

Manganese, which is grouped with iron in chemical classifications, is a relatively infrequent component of water, and even when present is usually found in small amounts. It is more frequently found in ground water, but its occurrence in surface water has also been reported.

In water, manganese behaves somewhat like iron, but it usually oxidizes more slowly and with the formation of the black hydrated oxide instead of the familiar red iron oxide. Usually its presence is due to the decomposition of organic matter in manganiferous soils and the resulting solution of the manganese in the humic and other soil acids as well as in carbonic and sulphuric acids.

In the stagnant bottom waters of certain deep reservoirs, a condition which favors the production of hydrogen sulfide and the bacteriological decomposition of sulfates, manganese is often present, usually in combination with iron.

MANGANESE CONTENTS OF WATERS

The manganese contents of various waters are given in tables 1 and 2.

In New Jersey in 1928 the Department of Health (1) found that the manganese content in 238 of the potable water supplies of that state ranged from 0.03 to 0.20 p.p.m.

Corson (2) found that out of 147 waters from wells, 60 had a manganese content of from 0.03 to 2.8 p.p.m. Of ten spring waters, 6 contained no manganese, 3 contained 0.40, and 1 contained 7.8 p.p.m.; while 11 of the 16 streams contained from 0.02 to 0.09 p.p.m. of manganese.

Perhaps the most notable occurrence of manganese was the so-called "manganese calamity" of Breslau, Germany in the spring of 1906, which is described elsewhere (3). Here the river Oder rose and over-

[1] Of Weston and Sampson, Consulting Engineers, Boston, Mass.

flowed the well-field, increasing the iron content to 440 p.p.m. and the manganese content to 220 p.p.m. This load was too great for the deferrization plant, manganese passed the filters, and the supply was temporarily abandoned. At Breslau it was generally considered that during the preceding dry period the manganese in the peaty top soil had become dissolved as sulfate, which easily-soluble salt readily leached into the water-bearing layer below, following the high water.

TABLE 1

Ground waters—manganese contents

LOCALITY	MANGANESE
	p.p.m.
Amesbury, Mass.	0.30
Babylon, N. Y.	0.07
Barnstable, Mass.*	0.03
Bayshore, N. Y.	0.37
Billerica, Mass.	1.60
Bjornstop, Sweden	3.4 to 53.4
Breslau, Germany	Trace to 220
Brookline, Mass. (av.)	0.41
Calverton, N. Y.	0.30
Cohasset, Mass.	0.15 to 0.40
Halle, Germany	1.50
Kingston, Mass.*	0.05
McKees Rocks, Pa. (Pittsburgh Suburban Water Co.)	1.0 to 4.0
Medway, Mass.*	0.05
Middleboro, Mass. (av.)	0.67
Patchogue, N. Y.	0.20
Reading, Mass.	0.004 to 0.56
Stettin, Germany	5.22
Superior, Wisconsin	0.12

*Results by courtesy of Mr. H. W. Clark, Chief Chemist, Massachusetts Department of Public Health.

It is undoubtedly true that the presence of manganese in many waters would have attracted little attention were it not for its effect on the control of chlorine treatment by the ortho-tolidine test. Buswell and Boruff (4) found that manganese interferes with this test, and Enslow in discussing their paper gave examples thereof.

Forsberg (5) showed that manganese in surface waters reacted like chlorine; unfiltered waters gave a positive reaction for chlorine, but those filtered through paper to remove the manganese did not so react.

In New Jersey (6) considerable difficulty has been experienced because of this interference of manganese with the ortho-tolidine test. In practice, when the manganese fluctuates, the "chlorine blank" is interfered with and the determination of the proper dose of chlorine is made difficult.

TABLE 2

Surface waters—manganese contents

LOCALITY	MANGANESE
	p.p.m.
Amherst, Mass.*	0.01
Amsterdam, Holland	0.30
Baltimore, Md.	0.20 or less
Baberton, O.	0.90
Boonton, N. J.	0.04
Boston, Mass. (Metropolitan)*	0.01
Branchville, N. J.	0.08
Brockton, Mass.*	0.01
Burlington, N. J.	0.10
Butler, N. J.	0.04
Columbus, Ohio (Scioto River)	0.22
East Liverpool, O. (Ohio River)	0.88
Jersey City, N. J.	0.04
Marlboro, Mass.*	0.04
New Bedford, Mass.*	0.03
New York City (Croton System)	0.03 to 0.10
Ogdensburg, N. J.	0.06
Oakland and Berkeley, Cal.	Less than 0.10
Phillipsburg, N. J.	0.04
Rahway, N. J.	0.03
Springfield, Mass.*	0.14
Sussex, N. J.	0.08
Wanaque, N. J. (Wanaque Reservoir)	0.00 to 2.5
Youngstown, O. (Meander Creek)	0.22

* Analyses by courtesy of Mr. H. W. Clark, Chief Chemist, Massachusetts Department of Public Health.

There are also others who have observed the interference of manganese with the ortho-tolidine test, and the following phenomena have come under the writer's observation.

MANGANESE IN THE WANAQUE RESERVOIR

The Wanaque Reservoir is located in the Jersey highlands. It is designed to supply 100 m.g.d. to the populous district of north-

eastern New Jersey including the cities of Newark, Paterson, Passaic, Clifton, Montclair and Kearny.   At present a portion of the district, as well as the city of Bayonne in addition, is being supplied with an average of about 25 mg. daily.   Later in the year the cities of Paterson, Passaic and Clifton, and perhaps another city in the district, will begin taking water from this source.   The reservoir is in a favored location and stores 28,010,000,000 gallons of water, having the average chemical composition during 1930, its first year of use, shown in table 3.

The valley of the Wanaque is one which has been scoured out by glaciation and then refilled with glacial deposits.   At the dam site these deposits reach a depth of 110 feet.

TABLE 3

*Composition of Wanaque reservoir water**

CONSTITUENT	P.P.M.	CONSTITUENT	P.P.M.
Turbidity	5.0	Hardness	20.0
Color	13.0	Alkalinity	14.0
Oxygen consumed	3.6	Total residue on evaporation	40.0
Nitrogen as free ammonia	0.015		
Nitrogen as albuminoid ammonia	0.152	Suspended residue on evaporation	0.5
Nitrogen as nitrites	0.001	Carbon dioxide	1.5
Nitrogen as nitrates	0.05	pH value	7.0
Chlorine	2.0	Iron	0.16

* Analyses by the Wanaque Laboratory, A. E. Griffin, Analyst, supplied by courtesy of J. W. Griffin, Superintendent of Operations.

The terrain is hilly and the half-dozen feeders drain areas sparsely covered with soil.   Only 1.5 per cent of the flooded area was swampy in character.   The bottom of the reservoir was well cleaned, but was not stripped to more than 10 feet below the flow line excepting in the vicinity of the dam.

In the catchment area contributing to the reservoir are Sterling and Greenwood Lakes, the latter receiving the runoff from 27.1 square miles; the catchment area above the reservoir dam is 94.4 square miles.

The catchment area is a highly mineralized one on which were located some of the earliest American forges, the ruins of one now being beneath the waters of the reservoir.   Contributing to it is the

drainage from Peters' Mine, one of the existing iron mines of the Ringwood Company. This mine is in active operation and other mines in the vicinity have been worked intermittently for years.

The reservoir began to fill on March 23, 1928. It was full in March, 1929, drawn down 10 feet during that year, and refilled on March 4, 1930, shortly before it was placed in service.

With an average iron content of only 0.16 p.p.m. no attention was paid at first to manganese. Soon after, the "chlorine blanks" in the ortho-tolidine tests for chlorine began to vary unreasonably and proper dosage was difficult. This condition was first observed in May, 1930, when the bottom water was beginning to stagnate and to

TABLE 4

*Composition at various depths**

DEPTH BELOW SURFACE	COLOR	CARBON DIOXIDE	DISSOLVED OXYGEN, PER CENT SATURATION	MANGANESE	CHLORINE BLANK
feet	p.p.m.	p.p.m.		p.p.m.	p.p.m.
0	10	0.5	106	0	0.03
10	11		105	0	0.03
20	22	1.5	90	0.05	0.06
25	15	7.4	40	0.05	0.07
30	14		27	0.06	0.10
40	10	8.5	31	0.14	0.17
50	20		29	0.36	0.25
60	12		22	1.10	0.30
70	13		35	1.90	0.24
77	16	12.2	0	2.40	0.15

* Analyses by the Wanaque Laboratory.

affect gradually the water drawn from the intermediate intake, about 35 feet below the flow line of the reservoir. Later in the summer when the oxygen had disappeared and the carbon dioxide was high in amount, the trouble became serious.

The set of depth samples collected on July 4, 1930, shows the peculiar condition of the reservoir. Results of analyses of these samples are given in table 4

The source of the manganese was undoubtedly the bottom of the reservoir where oxygen was absent, hydrogen sulfide present and carbon dioxide abundant. Tests made recently show that the amounts of manganese in the feeders of the reservoir vary from 0.03 to 0.06 p.p.m., as compared with a manganese content of 0.13 p.p.m.

in the reservoir water. At this writing the manganese content is apparently decreasing rapidly as the effect of the spring overturn recedes and the reservoir clears. Consequently, troubles with the ortho-tolidine tests are absent for the present. Difficulties, however, are expected in the fall of 1931 and again in the spring of 1932. As the reservoir ages, however, it is expected that these troubles will become minimized.

While high amounts of manganese have been found during the past year, it is probable, as has been true elsewhere, that the manganese content of the reservoir water will decline as the fermentable organic matter in it decreases with the years.

While the effect of manganese on the ortho-tolidine tests is not the subject of this paper, it is one of vital importance for the sanitary control of the Wanaque supply. An indication of its seriousness may be gathered from the figures in table 3.

These figures show how easily one can mistake manganese for the chlorine demand of the water itself (chlorine blank), especially when water is taken from the lower half of a reservoir during a period of stagnation and high manganese.

At the Wanaque reservoir there is no fixed relation between the chlorine blank and the manganese content, as reported elsewhere.

It will be noted in table 3 that the chlorine blank is about equal to the manganese content at a depth of about 20 feet. This relation persists until a depth of 40 feet is passed when the manganese content increases more and more with each increment of depth. This may be due to variations in the degree of oxidation of the manganese, although such a statement is made with reservations. It will also be noted that the chlorine blank fails to increase below a depth of 60 feet. This is because of the counter influence of ferrous iron, as unpublished work of the Wanaque laboratory has shown.

So far no entirely satisfactory method of overcoming the disadvantages in estimating the dose of chlorine has been found, although the suggestion that the addition of ferrous sulfate might overcome the disabilities of the tests has been given a trial, so far without advantages.

MANGANESE IN INDUSTRY

At Baltimore, Baylis (7) found that manganese interfered with the bleaching of yarn, a condition described by Goldthwaite (8) of the Mellon Institute of Industrial Research. Kneeland (9) reported

trouble in laundries supplied by the Pittsburgh Suburban Water Company with water taken from beneath the Ohio River at McKees Rocks, Pa. This water contained between 1.0 and 4.0 p.p.m. of manganese with only a trace of iron. It had a hardness of about 190 and contained some Crenothrix. The manganese is retained by the zeolite softeners without damage to the zeolite which is made from Glauconite.

At Biltmore, N. C., a bleachery takes water from the Swannanoa River and treats it in a modern rapid filter plant. Just above the intake of the plant was a new dam forming a reservoir of considerable size. Soon after starting the plant there was enough manganese in the filtered water (about 0.3 p.p.m.) to stain the walls of the filtered water basin and collect on the rubber squeeze-rolls in the washing machines. While this condition did not affect the cloth greatly, it caused considerable apprehension. It was overcome by adding 0.5 p.p.m. of chlorine to the raw water. The manganese in the raw water has decreased since the plant was first started three years ago, indicating unquestionably that its presence was due to the solution of the metal from the rich, organic soil in the bottom of the new reservoir.

MANGANESE IN PIPES

Manganese frequently coats the inside surfaces of aqueducts. Hale (10) described the coating of the Catskill aqueduct with manganese and concomittant substances and stated that it was induced by the alkaline reaction of the inner surface of the concrete aqueduct. On the other hand, a coating which was high in manganese appeared in the Wanaque aqueduct, which is of steel, after a few months of use. Hale found that the application of a considerable dose of chlorine not only prevented the formation of new coatings, but loosened the old ones and caused them to slough off. The coating found on the Wanaque aqueduct was derived from a water which was always treated with a low dose of chlorine. The difference in observed effects is due probably to a difference in dosage, a small dose of chlorine in the case of the Wanaque aqueduct being insufficient to dislodge the manganiferous coating.

The writer feels confident that in the soft waters of the Atlantic seaboard, at least, either physico-chemical reactions or the presence of bacteria, including the manganese bacteria, may cause deposits of manganese in aqueducts and pipe lines, and it seems that whether or

not coatings are formed depends upon a great many factors, among which are reaction, organic matter, organisms and iron content, as well as concentration of manganese.

DEMANGANIZATION

The methods for the removal of manganese parallel those for the removal of iron, although they are not so simple, and emphasis must be placed on certain factors which in deferrization plants are of less vital importance, because reactions are more rapid.

It may be stated as a basic principle, that manganous salts in alkaline waters absorb oxygen, oxidize slowly, and ultimately precipitate as manganic hydrate (hydrated manganic oxide). There seems to be no difficulty in the oxidation of manganous bi-carbonate or sulfate to the hydrated manganic oxide. The difficulty is in removing the latter from the water, or in absorbing the manganous salts and holding them until they may be oxidized.

That precipitation is relatively slow is shown in systems supplied with untreated ground water, from which small amounts of iron and manganese collect in the distribution system. For example, at Brookline (Mass.) (11), before the deferrization plant was placed in use, the ratio of manganese to iron in pipe coatings increased with the distance from the pumping station along the force main and the pipes in the distribution system.

That organisms, Crenothrix and the manganese bacteria, play an important rôle in the removal of manganese from water, has been shown by von Wolzogen Kühr (12), Tillmans, Hirsch and Grohmann (13) and Janzig and Montank (14).

Von Wolzogen Kühr not only attributes the absorption of manganese salts from water to manganic oxide, but states that manganese bacteria play the chief rôle in oxidizing the manganese, which they do by biolysis. This point is still in controversy, and the writer's experience would incline him toward the opinion that the bacteria and the manganic hydrate which they accumulate, absorb the manganous salts and retain them while they are being slowly oxidized and changed from the colloidal state. One may not disregard the effect of the mass of precipitated manganic oxide with which the water comes in contact in filters and tricklers; neither the mass of zoögloeal jelly produced by bacterial growth.

If the biolytic action of bacteria were the essential agent in oxidizing manganese, the application of killing doses of chlorine at Bilt-

more, N. C., would have decreased rather than accelerated the removal of manganese by the mechanical filters there.

That the accumulations of bacteria on filtering material are important, is not to be denied. It was demonstrated at Brookline, Mass. Here, during the first months of operation of the deferrization plant, most of the manganese passed the tricklers and was partly removed by the filter sand, which was stained black. At present, all of the manganese is removed in the tricklers, through which the water passes after being aerated, and before being filtered. During 1930, this water contained an average of 0.65 p.p.m. of iron, 0.41 p.p.m. of manganese, and had a hardness of 39 p.p.m.

It may be true that oxygen and other gases produced by bacterial growth are more active than the same gases dissolved in water. If so, may not the increased effect be due to the greater concentration of the "nascent" gas at the point of liberation. In other words, the phenomena of surface contact and concentration are the important ones in demanganization.

The devices commonly used for treating waters for the removal of iron are also used for the removal of manganese, but preliminary treatment is more important in the latter instance.

In the softening process, manganese is readily removed by lime with the aid of ferric sulfate. Hopkins (15) found at Baltimore that the pH value must be maintained at 9.4 or higher to bring about the chemical precipitation of manganese with lime and ferric sulfate (chlorinated copperas).

As in the deferrization process, the manganese must be oxidized, coagulated, and the insoluble hydrated oxide removed by filtration. The coagulation of the small amount of manganic hydrate in dilute solution is the difficult step in the process.

Organic matter often interferes. Here pre-chlorination is sometimes advisable, but not to a degree sufficient to inhibit growths of organisms on the surfaces of the filtering material, tricklers and filters.

While hard waters, high in iron, may be purified easily, soft waters, low in iron and pH value, and high in organic content, present many difficulties. Some of these (e.g., Lowell, Mass.) may not be aerated to a high degree for fear of forming a resistant compound of manganese and organic matter. Others require the addition of an oxidizing agent, like chlorine or permanganate, to reduce the inhibiting organic matter.

Generally speaking, it is better to depend on surface contact with accumulations of hydrate and organisms, than upon overdosing with alkali, with the resulting increase in hardness of soft water. Aeration, contact with precipitated hydrate and organisms in a contact aerator, followed by filtration through sand, suffice to remove manganese from water.

While it is true that manganese zeolites, regenerated with sodium or potassium permanganate, may be used successfully, the methods in more common use are usually preferable. The devices designed for iron removal, with allowance for the slower oxidation of manganese, will meet conditions as a rule.

Excepting in certain cases, aeration should be thorough. Either high-head or low-head nozzles, cascades or a system of superimposed coke or gravel-filled trays may be used effectively. The latter system has the advantage of providing a larger contact surface.

Contact aerators or tricklers should be deep enough to provide ample time of contact. The present tendency is toward the upward-flow type as used by Pirnie at Long Beach (16). This is because of saving in head, greater facility for cleaning, and a longer period of contact. Rates of from 50 to 80 m.g.a.d. are usually satisfactory,—choice of rate depending upon the character of the water treated.

Storage of water before filtration is usually beneficial, especially with gentle agitation, and in many cases, storage with stirring may take the place of contact bodies. At Brookline, Mass., F. F. Forbes arranged to have the trickler effluent pass through narrow channels at a moderate velocity, on its way to the filters. The results of this arrangement have been excellent.

For manganese, slow filters, operated at a rate of as high as 10 m.g.a.d., are usually safer than rapid ones, although with ample pre-treatment, the latter may be used. In general, if the preliminary treatment be adequate, high rates of filtration may be used.

Manganese in water is a public nuisance, and its removal adds greatly to the comfort and enjoyment of consumers. It is fortunate that demanganization, like deferrization, may be accomplished so readily, and at such reasonable costs.

REFERENCES

(1) Report of the Department of Health, New Jersey, 1929.
(2) CORSON: University of Illinois, Bulletin 5 (1916), *14*, 169–172.
(3) WESTON: Jour. A. S. C. E., *64*, 175.

(4) BUSWELL AND BORUFF: Jour. A. W. W. A., *14*, 384 seq.

(5) FORSBERG: Jour. A. W. W. A., *15*, 706–708.

(6) FORMAN: Jour. A. W. W. A., *21*, Sept. 1929.

(7) BAYLIS: Jour. A. W. W. A., *12*, 211 (1924).

(8) GOLDTHWAITE: American Dye Stuff Reporter, Oct. 23, 1927.

(9) KNEELAND: Jour. A. W. W. A., *13*, 436–438.

(10) HALE: Jour. A. W. W. A., *22*, 462.

(11) WESTON: Jour. N. E. W. W. A., *28*, 27.

(12) VON WOLZOGEN KÜHR: Jour. A. W. W. A., *18*, 1–23.

(13) TILLMANS, HIRSCH AND GROHMANN: Gas u. Wasserfach., *71*, (1928).

(14) JANZIG AND MONTANK: Jour. A. W. W. A., *21*, 1319 et seq.

(15) HOPKINS AND WHITMORE: Jour. Ind. & Eng. Chemistry, *22*, 79.

(16) Water Works Practice, 249.

DISCUSSION

MR. WEED of Harrisburg called attention to the fact that at that plant, correction of corrosive tendencies in the water by treatment of the filter effluent caused the precipitation of unsuspected manganese. The addition of soda ash prior to or with the coagulant, alum, caused the manganese to be removed in the filters.

MR. C. P. HOOVER, Columbus, Ohio, suggested that in waters that are relatively hard, the use of lime will remove manganese. Thus in softening plants, its presence offers no problem.

FILTERING MATERIALS FOR WATER WORKS

By Wm. E. Stanley[1]

The present active interest in the question of filter materials for water work, coupled with some dissatisfaction with the present status of our knowledge and the standards in use, indicate the desirability of more extended consideration and research covering the relative effects of various characteristics of water works filter materials.

A committee of the Sanitary Engineering Division of the American Society of Civil Engineers has been giving attention to filter materials for water works for the past several years as one phase of its activity.

The objective of this paper is to indicate something of what has been undertaken and to outline briefly some of the factors and problems involved, to point out some lines of study which may be profitably developed and to call attention to the need and value of coöperative effort among the various investigators who are or may be conducting active studies. Mr. Armstrong and others to follow will present a number of pertinent data from several series of experimental work which will undoubtedly shed some light on certain of the problems involved.

The more common filtering material for water works is a bed of well graded silica sand supported, generally, by a bed of gravel. Occasionally some other material is used for the filter media as, for instance, crushed coal at Denver. For the present discussion the filtering material is assumed to be graded sand.

FUNCTIONS OF THE FILTERING MATERIAL

There are some differences of viewpoints as to the comparative merits of various characteristics of filter sand. Accordingly it seems desirable to review briefly some of the more fundamental functional factors.

The operation of the filtering material in a rapid sand filter presents an entirely different situation from that in a slow sand filter. In the former the rate of filtration is, of course, much greater, but it is the hydraulic grading during the backwash which makes the main

[1] With Pearse, Greeley and Hansen, Consulting Engineers, Chicago, Ill.

difference in the properties of the filtering materials in the two types
of water filters.   Since our principal standards for filter sand, namely,
effective size and uniformity coefficient were originally developed in
connection with the slow sand filter, possibly some modifications in
such standards might be obtained to fit the conditions better which
arise in the operation of the rapid sand filter.

The present yardsticks of effective size and uniformity coefficient
are so generally accepted that any proposed change must be con-
sidered with great caution.   The investigator will do the engineering
profession a service if he will present his results in terms of the present
as well as of his proposed new standards.   It is all well enough to
say that the present standards do not apply.   It is another thing to
discard them unceremoniously in favor of some new standard which
may be based on a minimum of data or possibly only on ideas.   Any
new hypothesis will be accepted grudgingly and will be subjected to
analysis from many angles undreamed of by its originator and pro-
ponents before it finally becomes an established standard.

Since the primary function of the filter units of a water filtration
plant is to remove as completely as possible those impurities re-
maining after preliminary treatment, the filtering material should
have such qualities as to perform this function most effectively and
economically.   Actually the service required of the filtering material
is more complex than that of a mere strainer.   It should perform as an
efficient strainer, frequently under wide variations in quality of ap-
plied water.   On the other hand, it is desirable that there be as low a
resistance to flow as possible and it is necessary that the filtering
material have such characteristics as to permit it to be readily
cleaned during backwashing.   These requirements are contradictory
in effect.

The filter units cannot be dissociated from the other elements of a
filtration plant, nor can the investigation of filter sand properties be
completed without orienting those properties with the work to be
done by the treatment units ahead of or following the filter units.

The acceptance of chlorination as a safe and practical method of
bacterial removal has reduced the function of the filter bed to that of
producing a clear water.

With present pretreatment processes, there are times when it is
not possible with many waters to obtain a strong flocculation of the
coagulated material.   During such seasons the coagulated material
passes more readily into and through the filter bed and the filter run

may be determined by the clarity of the effluent rather than by the loss of head. This condition creates the necessity for any study of filtering material to include some consideration of pretreatment processes and particularly of the seasonal or geographical variations in character of water, insofar as such variations are related to the conditions under which the filter media must function.

There is undoubtedly a very definite relationship between sand size and gradation to wash water rates and hence to the hydraulic capacities of the wash water system. Thus a study of filter material cannot be complete without considering the relationships between sand characteristics and the structural design of the filter plant as controlled by wash water capacities, underdrain operation and the like.

EFFICIENCY OF THE FILTER BED

The efficiency of a filter bed might be taken as the ratio between the actual output of the filter to the theoretical output, except that in ordinary practice it is not necessary to operate a filter to its maximum capacity continuously. The filter bed in a water plant is not a simple unit as regard either the input or the output. The output may be measured by the quantity and clarity of the water filtered per unit of filter bed or per volume of filtering material. This should be done at a minimum of input of energy and dollars.

The amount of filtered water required to keep the filter bed in a clean condition is perhaps the best measure of the major loss in the operation of the plant. This is determined largely by the frequency with which the filter must be washed. Thus the length of filter run is pertinent.

The length of filter run may be determined by one of two factors. First and most common is the factor of loss of head or resistance to flow, which is built up in the filter. In some instances the filters may have to be washed more frequently because of the passage of flocculated materials. This may occur more readily with large sized filter material and the occurrence frequently is more pronounced during certain seasons of the year when troubles with weak coagulation are experienced.

Improper cleaning of the filter bed and the development of cracks, mud balls, condensed areas, and other troubles frequently reduce the actual surface available for use, thus decreasing the filter output and the efficiency of operation. In the final analysis, the quality of the

effluent must be the measure of effectiveness of the filter units and other items such as the amount of wash water and the effective capacity of the bed are secondary.

FILTER SAND PROBLEMS

The problems which occur in connection with filter sand for water work may be those which relate to matters of design, which refer primarily to the question of proper physical characteristics and quantities to be specified and included in the construction of filters, and those which occur in plants after they have been put into operation. These relate particularly to various abnormalities in behavior of the filter sand under operating conditions.

There is a very definite relationship between the two sets of problems. The aim of the designer should be to provide a filter material with proper physical characteristics and in proper quantities to the end that the operating troubles will be a minimum.

There is a fairly well defined agreement as to the standard or normal rate of loading which can be applied to a given area of filter bed surface. There is, however, some uncertainty as to the quantity of filter material required as determined by the depth of the filter bed. It is not unlikely that the present practice might be modified as more definite information becomes available on the causes of troubles and the effects of various physical characteristics, such as size and gradation of filter sand.

Generally speaking, there is little difference of opinion as to what constitutes a desirable physical quality in filter sand. There is some evidence to indicate some difference in hydraulic properties as between a sand with sharp angular grains and one with rounded grains.

The principal question with reference to the design of sand beds is that of the proper specification for the size and gradation of the filter sand. Mr. Fenkell has recently aptly stated that "the best specification for filter sand is still to be written."

The terms effective size and uniformity coefficient have been used for so long that many water works men and material supply companies use them as a matter of course and probably with little or no understanding as to the exact meaning of the terms.

It has been frequently pointed out within the last few years that the actual effective size of the filter sand in a rapid sand filter may be the top few inches or possibly fractions of inches of the sand bed. In recent months a considerable amount of experimental work has been

conducted on the relationship of the hydraulics and the variation in the shape and size of the sand grains of the sand bed.   Thus Professor Fair of the Department of Sanitary Engineering at Harvard has been directing a study relating to the hydraulics of sand filtration.   At Detroit an extended study resulted in the conclusion that the 30 percent size was the more logical size from the point of view of the hydraulic properties of the filter bed.   The Detroit experimental results were published in 1929 and present a very valuable addition to our knowledge of filter bed operation.

Another item under the problem of design relates to the question of the determination of size by sieving and the proper method of recording the results of the sieve analysis.   This item has been the subject of study by a special committee of this Association for a number of years and much improvement has been obtained in the facilities for analyzing and recording size characteristics of filter sand. Undoubtedly there is considerable yet to be done before really accurate methods are available, particularly in connection with the more fundamental researches.

Proper sand depth has been given little attention in the researches which have been made.   The practice of specifying a 30-inch depth of sand is based on precedent rather than on results of experimental work.   In a number of cases the depth has been reduced to 24 inches. It is probable that, with more definite understanding of the relationship between various sands and the type of water to be applied to the filter, some economies in construction may be obtained by the use of less depths of the carefully selected filter media.   It is also possible that a larger amount of supporting material which can be obtained from cheaper or local sources may be used with a relatively small or thin layer of carefully graded filter sand.

The problems of operation relate primarily to the inconvenience to the operators, the reduction in capacity, and the increase in costs of operation due to various diseases or abnormalities in the behavior of the filter sand.   One of these is the shifting or intermingling of sand and gravel which occurs occasionally in filter installations.   This shifting of the sand and gravel may be due to improper selection of sand and gravel or improper design of the wash water system which permits excessive velocity of water to be applied to the gravel and sand bed with such force that the gravel is lifted and mixed with the sand bed.

Another operating problem is that of the influence of biological

growths within the filter bed. These growths may become so enlarged that the character of the surface of the sand is considerably deteriorated. These difficulties can be reduced by some means of keeping the sand clean. There is considerable disagreement, however, among water works operators as to the relative value of increased washing rates and other methods such as violent agitation of the surface of the sand during or prior to backwashing. The experience at Detroit and Cleveland tends to suggest wash water velocities somewhat higher than the ordinary 24 inches rise or 15 gallons per minute per square foot of area wash water rate.

In some plants difficulties with incrustation incident to pretreatment and sometimes cementation have been experienced. There may be some relation between the proper sand size to be used in a given installation and the tendency for incrustation. Thus if the incrustation occurs at a relatively slow rate the use of a fine grained sand in the beginning with some replacement of sand from time to time might prove more economical than to provide necessary modification in pretreatment to prevent the encrustation.

EXPERIMENTAL PROJECTS

Much of our present practice in the design of water filtration plants is based on the results of a limited number of experimental projects.

An extended series of experiments has been in progress at the 68th Street experimental filtration plant in Chicago, and a considerable investigation, including experimental work particularly with reference to sedimentation has been made at Detroit.

The Committee on Filter Materials of the American Society of Civil Engineers in 1927 started a program of experimental work on filter sands to determine the relationship between size of sand and depth of bed to the characteristics of water found in various places. In order to obtain results under as wide a range in character of water as possible arrangements were made to have experimental filter units operated in a number of cities rather widely distributed and selected on the basis of differences in water. These experimental units included small glass tube filters with sand ranging in depth from 21 to 30 inches, and in size from 0.40 to 0.62 mm. A total of 14 cities indicated their willingness to cooperate in these experimental studies to the extent of purchasing the necessary equipment and building the experimental units. All of the sand required for these several installations was graded by Mr. Armstrong and furnished to each city, in order to reduce one of the variables in the problem.

Instructions for operating the test filters were furnished each plant operator so that some uniformity of routine might be obtained.

The results of these tests have not been one hundred percent favorable. Many of the cities have found it impossible or inconvenient to operate their experimental unit. Five or six of the units have actually been operated through the series of tests outlined by Mr. Armstrong. Although the results of the tests of the coöperative filter units were not as complete as would be desired or as favorable as was expected the tests brought out sufficient evidence to warrant the effort expended. A number of difficulties developed which can be overcome by minor adjustments in the test procedure. Mr. Armstrong probably will enumerate these. It is the writer's conviction that a program of tests by miniature filters in a considerable number of filter plants will develop results of considerable value to further study and the work already done should not be overlooked. The data obtained and experiences should be useful in any further work.

FURTHER EXPERIMENTAL WORK

Apparently two major lines of study might be undertaken in experimental work on filter sand. One of these relates to the hydraulics of various sands and sand characteristics under clear water conditions, which can undoubtedly be developed by experimental work in hydraulic laboratories, and includes, as one element of the work, greater refinements than at present available in the measurements and recording of filter sand size and size characteristics. Such tests as those now under way under the direction of Prof. Fair will develop fundamental data along this line.

The second type of experimental work includes the operation of actual filtration units with coagulated water applied to the filter surface. Logically this type of experimental research should follow the more fundamental hydraulic studies under the preceding type of experiments. However, filtration plants must be constructed and operated and it will be a considerable time before the determinations and analysis of results of experimental work are completed. Thus practical requirements indicate that it may be necessary to conduct the two series of experimental work at the same time.

I understand from Mr. Baylis that some of his test results indicate a considerable difference in the length of filter run for tests when different methods are used for washing the filters. These

results emphasize the need in the operation of experimental filters of extreme care in the methods of washing and in other items entering into the operation of the test filter units. The work should also be done by a filter plant operator having sufficient knowledge and skill to understand what is happening in the operation.

The proper interpretation of the results of operation of experimental filters is quite difficult and uncertain because of the many factors which affect the operation of a filter unit. It would seem that the proper procedure would be to definitely record all of the data obtained and the conditions under which the tests were made so that the results would be available in later years for further analysis in the light of more complete information concerning some of the fundamental characteristics of the filtering material.

EXPERIMENTAL APPARATUS AND METHODS

There is some skepticism as to the value or reliability of the results of small glass tube filters, "pea-shooter" filters as they have been dubbed by one prominent water works engineer. A considerable data on comparative operation of the test tube filters and full scale filters, by Baylis at Chicago, by Armstrong at Baltimore, by Hering and Hulbert at Detroit and others seem to show that the small test tube filters have a definite place in any extended study of filter sand. The comparatively small expense involved in setting up and operating the small filters permits extending the scope of the tests sufficiently to bring out more definitely the effect of certain variables. Thus at Chicago Baylis has been operating a battery of glass tube filters with sand size ranging from about 0.2 to 2.2 mm., which would be quite a wide range to use for larger scale filters. The results obtained will at least give rather definite indications, if they do not entirely establish certain of the fundamental relationships between sand size and certain other of the variables such as length of filter run or loss of head at which floc will pass through a given depth of sand and the like.

At the Chicago experimental filter plant, three sizes of filter units are in use including many of the small glass tube test filters with a surface area of about $2\frac{1}{2}$ square inches, ten or more larger test filters with a surface area of 10 square feet, and two plant scale filter units each with a surface area of 100 square feet. Mr. Baylis has published some results of the operation of these three sizes of filter units, particularly with reference to the length of filter run and the increase

in loss of head during the length of filter run, which data definitely indicate that the small filter units give results comparable to those of the large filter units.

One question which has arisen and has caused considerable discussion in connection with the work of the Committee on Filter Material is that of the need of rate controllers on the experimental filters. The concensus of opinion seems to be that rate controllers are desirable.

Any program of investigative work must include careful sieving and recording of the sand sizes used. Present methods of sieving and recording sand sizes lack sufficient refinement for research purposes.

NEED OF COÖPERATIVE EFFORT

There is a need for greater coöperation between the designer, the filter plant operator and the various experimenters in filter sand operation. Such coöperation would be beneficial to all concerned. The type and characteristics of a filter plant and the filtering material are determined largely by the plant designer and it is up to the operator to make the best of the equipment furnished to him.

The experiences of different plant operators will be more valuable as the various operators have an opportunity to profit by the experiments and experiences of others.

Much of the value of the results of extensive and carefully developed investigations often is lost because of unwarranted assumptions as to some fundamental consideration, or by certain peculiarities of viewpoint of the person conducting the investigations. Any plan for reducing the effects of the personal equation would be helpful.

FILTER SAND

By James W. Armstrong[1]

The experiments which form the basis of this paper were undertaken in connection with the work of the Committee on Filtering Materials of the Sanitary Division of the American Society of Civil Engineers.

Before starting this work, if the writer had any ideas on filter sand at variance from those ordinarily accepted, they might be summed up as a growing opinion that sand should generally be a little coarser, wash water should be applied at a little higher rate, and that the size of the sand in the very top surface of the bed was the size that determined the value of the filter. Some of the facts observed and some of the data recorded, seemed to lead into wider fields than those which the Committee originally conceived to be part of its program. Such investigation could very properly be carried on by others whose interests lie within the field of theoretical research.

There has been an effort in this paper to reappraise old ideas, to clarify my mind as to just what takes place in the whole filtration process, and if possible, to evaluate some of the observations made in connection with the recent experiments. The opinions expressed do not in any way represent those of the Committee, but are entirely my own and embody the results of an effort to secure a fresh understanding of the subject. It is realized that the facts and conclusions stated cannot be considered general, as they might be very much modified if water of a different character should be used.

DESCRIPTION OF APPARATUS

The experiments were conducted by means of seven small glass filters made of Pyrex tubes about 5 feet long and about $1\frac{3}{4}$ inches average internal diameter. The tubes were closed at top and bottom by rubber stoppers and water was admitted at the top through a glass tube passing through the stopper. It was withdrawn from the bottom in a similar way. Each filter was provided with a mercury

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[1] Filtration Engineer, Montebello Filters, Baltimore, Md.

U tube gauge made of $\frac{1}{8}$-inch glass for determining the loss of head, and there was a similar gauge for indicating the head of water above the sand.　The influent pipe supplying the small filters was tapped
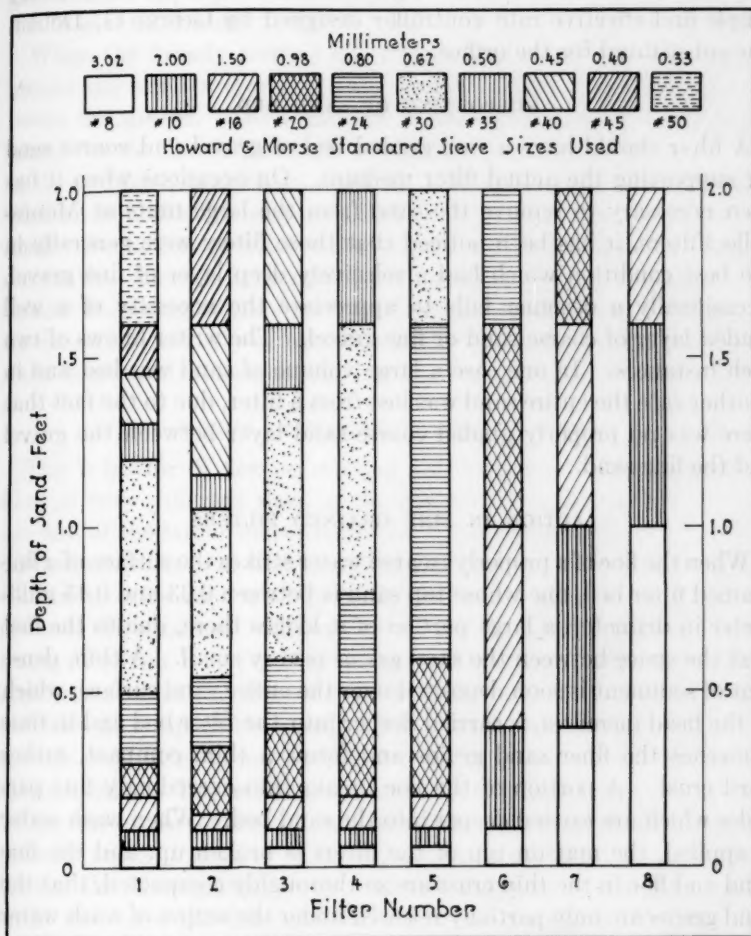

FIG. 1. DEPTH AND GRADING OF SAND USED IN SMALL EXPERIMENTAL
GLASS FILTERS

into the influent pipe of one of the large units.　By this arrangement water treated in the course of regular plant operation was supplied to each of the small units and there could be no question but that each filter received uniformly treated water, identical in every respect.

The filters were supported in such a way that it was possible to get a maximum loss of head of about $6\frac{1}{2}$ feet, but afterwards they were lowered to secure a maximum loss of from 8 to 9 feet. One set of runs was made with a fixed orifice for a controller. Later, a very simple and effective rate controller designed by George G. Dobler, was substituted for the orifice.

SUPPORTING OF SAND BED

A filter should have a well graded bed of gravel and coarse sand for supporting the actual filter medium. On occasions when it has been necessary to remove the sand from the large units at Montebello Filters, it has been noticed that those filters were generally in the best condition which had a relatively deep layer of fine gravel. Occasionally a designer fails to appreciate the necessity of a well graded layer of coarse sand or fine gravel. The writer knows of two such instances. In one case a large volume of sand was lost and in another case the entire sand was lost from a filter, due to the fact that there was no properly graded coarse sand layer between the gravel and the fine sand.

ACTION IN FINE GRAINED FILTERS

When the floc of a properly treated water strikes the surface of a fine grained filter bed, one whose top sand is between 0.33 and 0.45 millimeter in diameter, a large portion of it lodges there, due to the fact that the space between the sand grains is very small. A thin, dense mat of sediment is soon deposited over the entire sand surface, which as the head increases, is carried deeper into the filter bed and in time enmeshes the finer sand grains and forms a thin, compact, rather hard crust. A portion of the floc breaks into exceedingly fine particles which are carried deeper into the sand bed. When wash water is applied, the mat on top of the filters is broken up, and the fine sand and floc in the thin crust are so thoroughly compacted, that the sand grains are only partially released under the action of wash water and small matted pieces of sand and floc settle rapidly into the filter bed, where they remain and form the nuclei for mud deposits, which later give so much trouble in clogging of filter beds.

The floc adheres very tenaciously to the fine sand grains and cannot be entirely separated from them, even when wash water is applied at the very high rate necessary to lift the sand bed as much as 40 inches. The water between the sand grains does not appear clear, but con-

tains very finely divided suspended matter which is difficult to wash out. After the wash, as the sand settles back into position, small particles of floc can be distinctly seen adhering to the small sand grains in the upper portion of the bed. The reason for this is probably due to several causes.

When the loosely formed floc strikes the surface of the sand, it breaks and some of it sticks, while other particles enter the space between the grains. The smallness of the spaces offers little opportunity for the floc to penetrate freely into the interior of the bed, but as the surface mat thickens, the pull of the effluent tends to create a vacuum beneath the surface and to compact the floc still further. Under the action of wash water, the fine sand expands to such an extent that there is little rubbing of the grains together with the resultant loosening of the floc. As the space between the sand grains increases, the velocity of water necessary for carrying away the sediment is reduced; consequently, fine sand is not easily cleaned.

ACTION IN COARSE GRAINED FILTERS

The behavior of floc on striking the surface of a coarse grained filter is very different than in the case of a fine grained filter. On account of the larger spaces between the sand grains, there is a slower downward velocity of water and the floc settles lightly upon the surface or finds its way, without being broken up so finely, into the interior of the sand bed. The upper surface of the sand bed, even after a long filter run, is not closely compacted, but has a rather light fluffy appearance, and the floc can be distinctly seen in relatively large particles all through the sand bed. During the washing period, the sand is only lifted a few inches in height and is not closely graded hydraulically. The sand grains can be distinctly seen rising and falling under the action of the water jets which cause them to frequently strike together. There is a two-fold resultant tending to cleanse the filter quickly. The striking together of the sand grains tends to loosen the floc, and the low sand expansion causes a high velocity between the sand grains, which tends to take the loosened floc and carry it from the filters in fairly large particles. The water between the floc soon becomes clear, and after the washing period the sand settles back into position with apparently no floc adhering to it.

HYDRAULIC GRADING

It was noticed that the sand kept rising and falling in all the filters as long as the wash water was applied, but that in the coarser grained

filters there seemed to be an intermingling of sizes and there was continual striking or rubbing together of the sand grains. In the filters with the coarser grains the intermingling of sizes seemed to be more marked than in those with the finer grains.

In order to find out just what did happen, all the sand was removed from the tubes without disturbing its position. It was then divided into equal segments and the ones from the top, center, and bottom of the filter bed were dried and passed through a set of standard sieves.
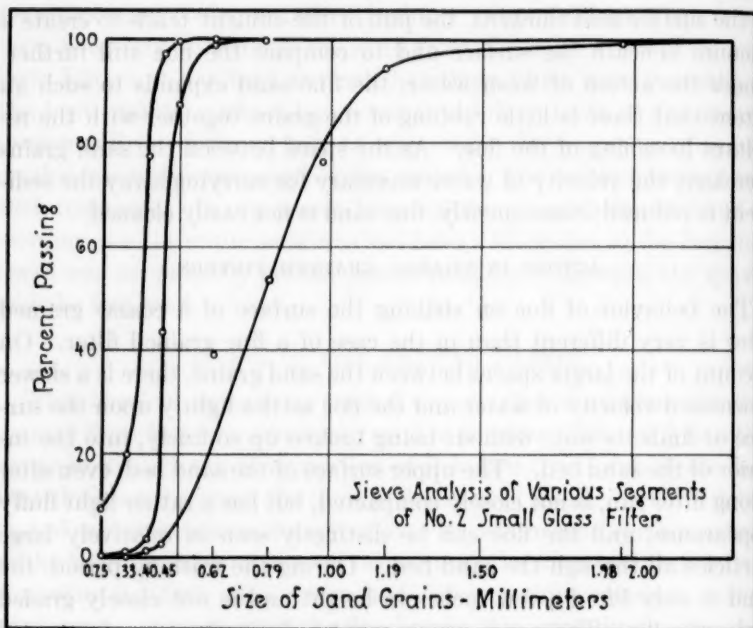
FIG. 2

Curves showing the result of the sievings were plotted and from these it was discovered that in filter 6 a mistake had been made by adding some smaller sand than it was planned to use in that particular filter. From the point of this particular test, it proved to be an advantage, as it revealed conclusively that the fine sand did not all rise to the top but that some of it was scattered throughout the bed.

A study of figures 2 and 3 shows that in the coarsest grained beds the hydraulic grading was very imperfect and that no close lines of separation could be distinguished between the different sizes, as with

the finer sands. (Probably the most unexpected discovery was the amount of coarse sand screened from the top section. The absence of close hydraulic grading may partly account for the long satisfactory runs obtained from very coarse grained filters.)

It may be that a filter bed composed of perfect spheres would grade hydraulically. With that thought in mind, different sizes of sand were examined under the microscope, as it was anticipated that the finer sands might be rounder, and if so, it would provide a theory
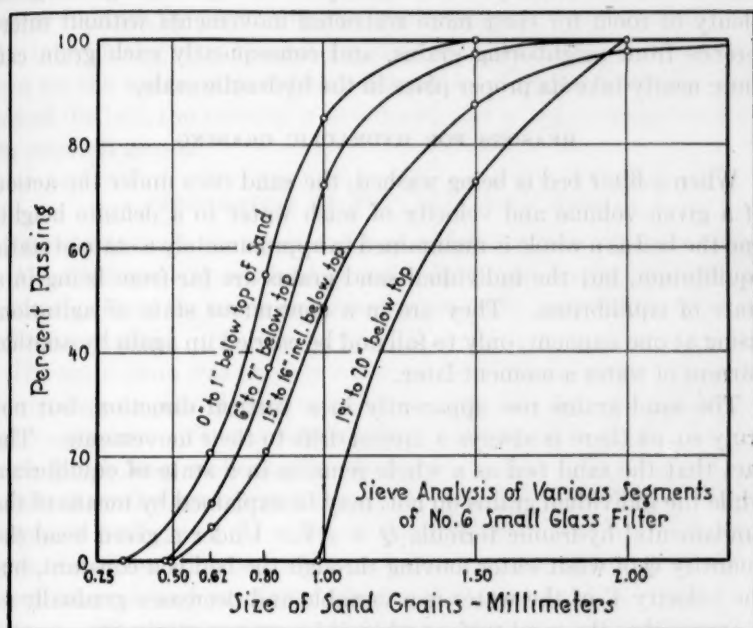
Fig. 3

for their better hydraulic grading. The examination, however, failed to reveal any smoother surfaces on the fine grains than on the large ones; in fact, they are relatively more irregular. (Why does coarse sand not grade hydraulically? Observations of the sand during washing suggest the following:

The jets or currents of water leaving the gravel surface are not uniform in direction or intensity. The sand grains when acted upon by a jet are lifted to different heights depending on their size and whether the surface acted upon is flat, round, or angular. When the

sand grain reaches the highest limit to which the jet can lift it, it is shifted to one side into a current of lesser velocity, when it settles down to the point where it is again picked up. In all these movements there is conflict with other grains, tending sometimes to lift them higher or to force them lower into the bed. The fine grains that have been lifted near the top are no longer subject to the same intensity of jet action because the greater sand expansion has increased the water area and the rising currents have been reduced to a more uniform velocity. The wide space between sand grains gives plenty of room for their more restricted movements without interference from neighboring grains, and consequently each grain can more nearly take its proper place in the hydraulic scale.

REASONS FOR HYDRAULIC GRADING

When a filter bed is being washed, the sand rises under the action of a given volume and velocity of wash water to a definite height, and the bed as a whole is maintained in approximately a state of static equilibrium, but the individual sand grains are far from being in a state of equilibrium. They are in a continuous state of agitation, rising at one moment, only to fall and be carried up again by another current of water a moment later.

The sand grains rise apparently in a vertical direction, but not truly so, as there is always a lateral drift to their movements. The fact that the sand bed as a whole remains in a state of equilibrium while the individual grains do not, may be explained by means of the fundamental hydraulic formula $Q = AV$. Under a given head the quantity Q of wash water moving through the bed is a constant, but the velocity V of the water is a variable and decreases gradually as it approaches the sand surface when it becomes a minimum.

In order to satisfy the formula $Q = AV$ and to maintain a sand bed in equilibrium, it is necessary that A, which is the total area in any given horizontal plane available for the passage of water, should become a variable and should increase as V decreases. In passing through the gravel, which is stationary, the water attains its highest velocity and leaves the gravel surface in small jets varying in direction and intensity. When the water enters a graded sand bed, which is floating, the jets strike the coarser grains forcing them upward to a point where the velocity of the jets is just sufficient to neutralize the force of gravity acting upon the sand. There is a constant struggle going on between the forces tending to lift the bed, and gravity which tends to pull the sand back to the bottom.

The hydraulic grading of the sand is due to the fact that the heavier grains require a higher velocity to sustain them. This velocity is secured by the pull of gravity on the sand grains, which reduces the total space between the grains available for the passage of wash water, to the area that will just satisfy the formula $Q = AV$.

As the jets travel upward they encounter other lighter sand grains which can be sustained by a stream of lesser velocity. A lesser velocity, however, can only be secured by a greater free area for the water to pass through. Gravity again comes in and preserves the proper balance by permitting the sand to expand more and more as the individual grains become smaller, thus providing the increased area for the water as its velocity decreases. Upon reaching the surface of the bed, the velocity is so reduced that it can no longer sustain the smallest grains.

PENETRATION OF FLOC INTO SAND BED

In order to learn the exact depth to which floc penetrated the filter bed and the amount retained in the bed, careful examinations were made of each of the seven experimental filters in accordance with the following method.

The moist sand was entirely removed from the tubes with as little disturbance as possible and immediately afterwards was divided into equal segments. The segments in the filters containing fine sand were made $\frac{1}{2}$-inch long, while in the coarse grained filters their length was increased as the penetration increased. Each portion was put into a separate beaker and the sediment thoroughly washed from the sand by means of a stirring rod. The sediment was then collected on filter paper, burned, and weighed. Curves of the results were plotted in figure 4. The penetration of the floc in inches was expressed as abscissa and the weight of sediment, expressed as per cent of the total weight, as ordinates.

A study of these curves revealed the fact that even after long filter runs the floc does not penetrate to a depth of over 2 or 3 inches for the fine sands, and that the penetration gradually increases until in the coarse grained filters the floc permeates the entire sand bed in a single long run. Some such behavior would naturally be expected but so far as the writer knows, no quantitative analysis has before been made to determine the capacity of various sands for receiving and storing floc. The significance of the facts revealed by these curves combined with other observations will be discussed under other sub-heads.

It might be concluded from an inspection of the curves that shallower filter beds could be used with fine sands. This would probably be true if filtration only were considered, but as fine sand facilitates the formation of mud deposits, it is doubtful if any filter should have a total sand depth of much less than 24 inches.

FILTER RUNS

In planning the tests, it was thought that the first two or three inches of the sand bed would be the controlling factor in determining the length of filter runs. Sizes were selected that appeared to be
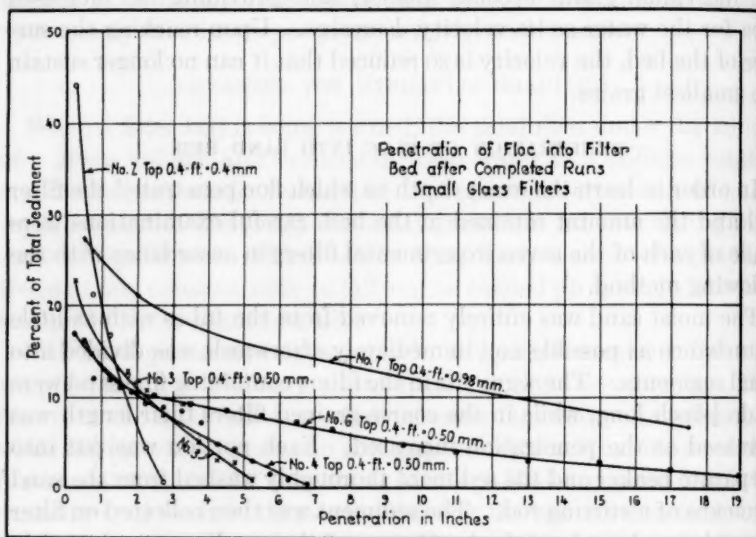
Fig. 4

absurdly fine at one end of the scale and absurdly coarse at the other end, so that curves could be plotted showing the full range of possibilities in filter sand. The top 0.4 foot in each filter was of uniform size and varied in the different filters from 0.33 to 0.98 millimeter in diameter. The sand supporting the top layer was not graded uniformly, as it had for the most part been used in a previous set of filter runs where a different top grading was employed.

In the earlier tests, runs were considered complete when the loss of head reached about $6\frac{1}{2}$ feet or the turbidity of the effluent reached 0.2 p.p.m. Later the loss of head was increased to 8 feet. In plotting

the results of the different runs, the finer sands showed characteristics very much as had been anticipated.   Nearly all the floc was removed in the top few inches and the runs were very much shorter than with the larger sizes. / The average of three runs of the finest sand was only 16 hours, whereas in filter 7 the runs averaged about 240 hours. The coarse grained sands gave such surprisingly long runs with a clear effluent, it was decided to replace the fine sand in filter 1 which gave runs too short for practical use, with sand having a top size of 1.5 millimeters.   To our astonishment the 1½ millimeter sand gave
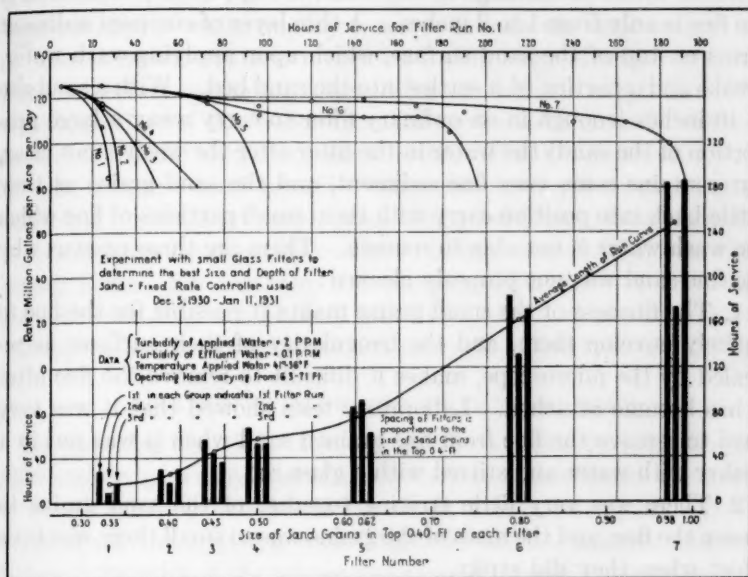
Fig. 5

surprisingly long runs before passing floc, although the applied water was unusually clear, around 2 p.p.m.   Ever since the beginning of these experiments it has become more and more apparent that sand much coarser than we had heretofore thought possible, could be used and still secure a good effluent.)

The question demanding an answer at this time is—what is the optimum size for filter sand? (For various reasons it is thought best not to name a size at this stage of our experiments, but two things seem to stand out clearly. (It will be much coarser than general practice has conceded to be desirable, and the deciding factor will prob-

ably not be the ability of sand to clarify water during the filtering
process, but the ability of the water to clean the sand in the washing
process. These matters will be discussed later in this paper.

OBSERVED BEHAVIOR OF FILTER SAND DURING WASHING

The effectiveness of a filter wash is dependent upon two things; the
velocity of the water in passing through the filter bed, and the size of
the sand grains. These facts can readily be determined by visual
inspection of small glass filters during the washing period.

In the filters containing the fine sand, the apparent penetration of
the floc is only from 1 to 2 inches.   A thin layer of compact sediment
forms on top of the sand surface, which upon applying wash water,
breaks and a portion of it settles into the sand bed.   With a sand rise
of 40 inches (enough in an ordinary filter to carry away a large pro-
portion of the sand) the water in the filter after the wash is not clear,
but contains some very fine sediment, and the sand grains as they
settle back into position carry with them small particles of floc which
the wash water is not able to remove.   There are three reasons why
the fine sand was not properly cleaned:

1. The fineness of the small grains makes it possible for the floc to
entirely envelop them, and the irregularity of their surfaces as re-
vealed by the microscope, makes it difficult to remove the floc after
it has become attached.   Laboratory tests showed that it was very
hard to remove the floc from fine grained sand when it was put in a
beaker with water and stirred with a glass rod.

2. There was very little striking together of the sand grains to
loosen the floc, and the mass of the grains was so small there was little
effect when they did strike.

3. The sand when lifted by the water remains in a state approxi-
mating equilibrium, and the expansion of the sand is so great that
the velocity of water passing the sand grains is not sufficient to cleanse
them of the adhering floc and at ordinary rates is not high enough to
properly remove the turbidity from the water.

The behavior of the filters containing coarse sand is very different.
The filter runs are naturally longer and produce an accumulation of
loose floc on top of the filter that does not form into a tough cake as
it does in the fine sand filters.   The floc in the filters with coarse sand
penetrates almost to the bottom of the sand layer, but is still retained
in the filter and does not pass into the effluent except in a very limited
amount after long runs.   Upon washing at a fairly high rate, the mud

cake on top of the filter is easily broken and does not settle into the filter bed. The sand does not grade hydraulically in any strict sense of the word as both coarse and fine grains are continually rising and falling. The floc passes out of the filters in large particles and after a short wash, the water above the sand is clear and the sand settles back into place without any floc adhering to it.

The large heavy sand grains require so great a velocity of water to lift them that they do not separate sufficiently to prevent striking or rubbing together, as they rise and fall under the influence of the water jets. The striking together loosens the adhering sediment and the high velocity of water quickly removes it from the sand bed.

From these observations it appears that in order to keep a filter bed clean, a certain minimum volume of wash water is necessary. The cleansing effect of this water upon the sand will be greatest with the sand that has the lowest expansion value. This makes it desirable to use the coarsest sand possible, consistent with good operating results, but it should be remembered that good filter design requires a proper balance between the results to be obtained and the cost of securing them. For instance, the advantage gained by a very high rate of wash may not be worth the added cost of the large pipes, valves, wash water gutters, etc. necessary to secure such a wash.

RELATION OF SAND RISE TO RATE OF WASH

By watching the glass tubes during the washing period, considerable knowledge was gained of the behavior of sand under the action of water that otherwise would have been difficult to acquire. It was noted upon first applying water, that the friction between the sand bed and the sides of the tubing was so great that it required unusual pressure and sometimes the jarring of the tube to break the matted sand. After the filter bed was thoroughly broken up, however, the movement of the sand under the action of water was apparently as free as it is in a large filter. Therefore, the observed action in the small units was taken as indicative of what happens in any filter.

A series of tests were made for each filter unit to determine the rate at which wash water should be applied to lift different sizes of sand to a given height above the filter surface. This was accurately determined by holding the expanded sand surface for one minute at a mark on the glass made at a definite height above the top of the sand before it had been disturbed. The water passing through the filter in this minute was measured in a one-liter graduate.

The curves showing the relationship between the rise of sand and the volume of water required to lift it, figure 6, indicate very clearly by the flattening of the curves, the well known fact that the volume of water required increases greatly as the grains increase in size, but the (curves also reveal what is not so well known, namely, that with a graded sand the rise is not proportional to the rate of applied water, but that it rises in a slightly greater ratio as the velocity increases) The plottings reveal a true curve and not a straight line as stated by Ellms and Gettrust[2] and Hulbert and Herring.[3]
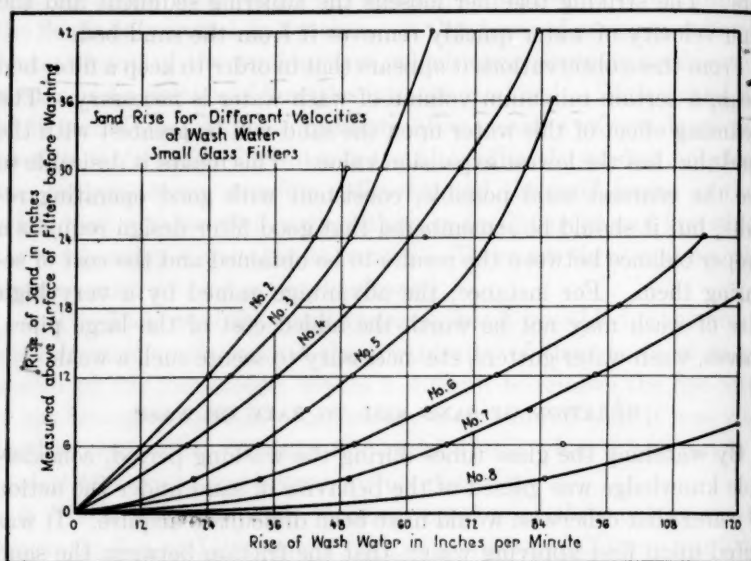
FIG. 6

The following reasoning is offered in confirmation of the observed data. In the experimental filters the top size of sand in each case is uniform and 0.4 of a foot deep. When wash water is first applied at a low velocity, a certain quantity will pass through the sand without lifting it at all, but as the velocity increases the sand bed is gradually lifted, beginning at the top. The formula $Q = AV$ may again be used to explain what happens. The top 0.4 foot being of uniform size is expanded just enough to secure an area between sand

[2] Trans. Amer. Soc. Civil Eng., 80: 1342.
[3] This JOURNAL, November, 1929, page 1445.

grains, in any given horizontal plane, necessary for passing the required volume of water at a velocity sufficient for holding in suspension grains of the smallest size. As all of the grains are of the same size, the same conditions of expansion hold for each successive plane throughout the entire 0.4 foot as expanded. The layer of sand just below the top being heavier, requires a greater velocity of water for lifting, and when the rate of applied water is sufficient for lifting this layer, the top layer has expanded still further and is pushed upward as a whole a distance equal to the expansion of the lower layer. When the third layer from the top has been lifted, the second and top layers have been still further expanded due to the increased velocity of water, and in addition the top layer has been lifted as a unit a distance equal to the total sand expansion of the two lower sand layers. This process continues as the rate of wash increases until the whole sand bed is held in suspension in accordance with the law acting to hold it in a state of equilibrium.

If the ratio of rise of sand to the rise of water be taken as the coefficient of sand expansion, it can readily be seen that each different size of sand has a different coefficient. If the coefficient of sand expansion be plotted as ordinates and the sand size as abscissas, the points will fall in a curve, which is a confirmation of the observed facts.

The pressure of water necessary for lifting the sand can be expressed in terms of head, and as $V = \sqrt{2\,gh}$, the formula $Q = Av$ may be expressed as $Q = A\sqrt{2\,gh}$ from which it can be seen that the velocity, and consequently the height to which the sand can be lifted, varies as the square root of the head, except as it may be modified by such factors as friction and the viscosity of the water. Therefore, the law of sand expansion is a true curve and with slight modifications is proportional to the square of the velocity of applied water. The reason that Ellms and Gettrust and Hulbert and Herring got straight lines for these curves may be due to the fact that the sand was of fairly uniform size and that they did not use very high rates of wash, and the true characteristics of the curves are not very apparent at the lower rates.

ESSENTIAL FACTS IN WASHING

There appear to be but two essential factors involved in the process of washing filter sand; the striking or rubbing together of the sand grains, which loosens the adhering sediment, and the scouring action

due to the velocity of the wash water in passing between the grains which removes the sediment and carries it from the filter.)

(The smaller the size of grains in a filter the greater will be the sand expansion for a given rate of applied wash water. The greater the sand expansion the further the grains will be separated and the less will be the chance of their striking together with sufficient force for loosening the sediment. The greater the space between the sand grains, the less will be the velocity of the water available for removing the sediment and the more difficult it will be to secure a thorough cleansing of the sand grains. Conversely, the larger the sand grains the greater will be the force with which they strike together, and the higher will be the velocity of wash water passing between them, and the easier it will be to keep the filter bed clean and free from mud deposits.)

RATE OF WASHING FILTERS

Tests made at Montebello Filters in the fall of 1929[4] seemed to indicate the desirability of using rates of wash higher than 2 feet rise per minute. Hulbert and Herring in their recent paper came to the same conclusions. Our experiments with the small glass filters also point very strongly to the desirability of using much higher rates than have been considered proper heretofore. In the early days of filtration, it was feared that a high rate of wash would displace the gravel layer; consequently, a number of the earlier filter plants were built with brass wire screens anchored to the floor system to prevent the displacement of gravel. Within the past few weeks I received a letter from an engineer stating he had again reverted to the same practice, as he had recently observed displacement of gravel in a number of filters.

About ten years ago at Montebello, tests were made on one half a full size 4,000,000 gallon filter unit which was equipped with perforated bronze strainer plates, to see whether or not it would be possible to displace gravel. These tests were made before the sand was placed in the filter, but with the complete 18-inch gravel bed in place. Under the severest conditions possible, which were obtained when water was applied suddenly at the rate of about 60 inches rise per minute, not the slightest permanent displacement could be discovered after a number of trials. With this knowledge in mind, it was very

[4] Edward S. Hopkins: Eng. News-Rec., 102: 841, 1929.

disconcerting to find at a later date, spots in some of the old filters, showing marked signs of gravel displacement, especially as it was not possible to wash one of these larger units at a rate exceeding about 30 inches rise per minute.   Shortly after putting the new slat bottom filters in service, it was discovered that on several occasions places in the sand bed had been blown up and at such times there was a loss of sand which was clearly indicated by a depression in the surface.   It was not until later that the real reason for the displacement in the old plant and the blows in the new plant became apparent.

It was noticed that these troubles seemed to occur at a place where a slight amount of entrapped air escaped through the filter bed. After a long filter run a crust is formed over the entire surface of the filter bed, which requires considerable pressure of wash water to break up.   If a small amount of air is permitted to escape through the filter bed in one spot, as can easily happen when air is permitted to enter one of the wash water pipes, this crust is locally broken and a weak spot is made in the filter through which wash water can easily pass.   Should the wash water valve be suddenly opened before the crust on top of the filter could be broken, a great volume of water will rush through the weak spot with sufficient velocity to displace the entire gravel layer within the area of weakness.   After this was discovered, orders were issued to filter attendants never to open a wash water valve wide until after the filter crust was thoroughly broken. Since that time, as far as the writer knows, there has been no further trouble with displaced gravel.

From the above it can be seen that with a well graded gravel bed and by using reasonable care in washing, there should be little danger of displacing gravel.   The desirability of using higher rates of wash is brought out elsewhere, but further experiments are necessary to determine just what rate is most desirable to properly cleanse different sizes of sand.

EFFECTIVE SIZE

The effective size of sand used in mechanical filter plants of this country averages about 0.4 millimeter and it varies in different plants from 0.32 to 0.6 of a millimeter in diameter. (It is believed that for the most part, precedent has been the determining factor in the selection of filter sand, but the variations from the average, indicate that there is still considerable difference of opinion as to the correct size.) The term "effective size" as technically defined, seems admirably

suited for sand as used in slow sand filters, but it is hard to see how the term can rationally be applied to a hydraulically graded sand, however incomplete that grading may be. A large volume of filter sand has recently been purchased under specifications using the term effective size, but it has been so hedged about with limiting qualifications, that it could have been omitted altogether without materially modifying the requirements of the specifications.

In view of the fact that with fine sands most of the work of filtration takes place in the size finer than the 10 per cent size, and with coarse grained sands most of the work is effected in the portion of the bed below the 10 per cent size, the term "effective size" should be redefined or some other term such as "top size" should be substituted.

FACTORS GOVERNING THE CHOICE OF FILTER SAND

This discussion assumes that filters are properly operated and that the water is correctly treated, as any other assumption would lead to uncertain conclusions. It is concerned only with these factors that are necessary for making a rational choice of sand best suited to all purposes of filtration. Two main problems are involved. One concerns the passage of water downward through the sand and the other concerns its passage upward through the sand. (The desirability of knowing the right size of sand for proper filtration has been recognized from the first, but the importance of having sand of a size that can be easily and quickly cleaned after filtration has not always been recognized.)

From the standpoint of filtration, it is desirable to have a sand that will (a) prevent any floc passing through the filters, (b) hold floc as loosely as possible in order to permit easy washing and prevent the formation of mud deposits, (c) hold as large a volume of floc as possible without clogging.

From the standpoint of washing, it is desirable to have a sand of such size that it will (d) cleanse itself and be free from adhering floc at the end of a wash, (e) permit the passage of water at sufficient velocity to remove all the sediment without losing sand.

In discussing the above points it will only be necessary to refer to the more extended discussions of other sections.

(a) A study of the results of numbers of filter runs shows that filters containing fine sand will not pass floc, neither will they pass water after a relatively short period of service. Filters with very coarse sand will pass floc, but only after a very long period of service,

runs sometimes lasting for over 250 hours. If the only criterion for the selection of sand were the passage of floc, fine sand would be most desirable.

(b) It was shown that in fine sand filters the floc tended to form in a dense hard mat on the surface and that when the mat was broken under the action of wash water, portions of it settled into the bed to form mud deposits, and that the floc adhered very tenaciously to the sand, making it difficult to wash. In coarse grained filters the wide space between the sand grains permits the floc to penetrate the sand bed without compacting and consequently it does not adhere as tightly to the sand. Coarse grained sand seems best considered from this point of view.

(c) The tests for penetration of floc into the sand bed, revealed that the coarse grained filters held very much more floc than the fine grained filters and that it was easier to remove. The ability of the sand to hold a large amount of sediment without clogging, means longer filter runs and less wash water. From this point of view the advantage is decidedly with the coarse sand.

(d) Observations of fine sand filters during washing show that with any rate of wash that will not waste sand, it is difficult to remove the fine suspended matter and that particles of floc adhere to and settle back with the fine sand grains. Coarse grained sand has a low expansive value which permits the rubbing together of the grains during the washing period. This rubbing loosens the sediment, and prepares the sand bed for quick and easy cleaning.

(e) In order to secure thorough removal of all sediment from the filter bed, it is desirable to pass water between the sand grains at high velocity. A heavy sand with low expansive value will permit the use of higher velocities with a given quantity of wash water, than a fine grain sand will.

For all reasons, except the possibility of floc passing through the sand bed, coarse sand seems best suited for the filtration of water, and as the larger grained filters give such very long runs without the danger of passing floc, there seems little reason for selecting fine sand. There seems to be a growing, well founded, conviction that high velocity washes are desirable, but should an unusually high velocity be used it would result in a greatly increased cost of construction. Large wash water pumps, tanks, pipes, valves, and gutters would be required.

From information gained by our experiments and from difficulties

encountered in the operation of filters, it has been growing upon me that the most desirable filter sand is one that can be easily washed. I am also of the opinion that filter sand, very much coarser than is generally considered desirable, should be used. The deciding factor in determining the optimum size of sand will probably be an economic one, as it will be necessary to preserve a proper balance between desirable operating results and the inevitable high construction and operating costs if a very high rate of wash water is used.

CONCLUDING REMARKS

It is believed that some of the opinions expressed are applicable to any filter, but that others might be considerably modified under different conditions. Further experiments are desirable for determining the best working limits of some of the factors involved in the problem, and for confirming or modifying the opinions expressed, especially as they may be affected by waters of different characters.

The highly colored, soft waters of New England, the Great Lakes waters, and the turbid waters of the Middle West should be studied under as nearly identical conditions as possible.

The Montebello experiments were conducted by George G. Dobler, Senior Chemist, and were reviewed by Edward S. Hopkins, Principal Sanitary Chemist.

FILTERING MATERIALS AT ST. LOUIS

By August G. Nolte[1]

The writer appeared on the scene of this experimental work after 18 of the 50 runs reported from the St. Louis Plant, had been completed. He is not intimately acquainted, therefore, with the difficulties encountered in the early stages of these experiments. However, all correspondence received from Mr. Armstrong has been studied and the brief discussion that follows is a result of the study given this matter under the aforesaid conditions.

The first filter or test run was started on June 6, 1929, and the 50th run completed on February 17, 1930. The object of the experimental work was to determine the best size and depth of sand in mechanical filters for any given water. These optimum conditions were assumed to be attained with the longest running filter giving the best effluent.

The variables in operating the experimental filters were:

> The length of run of the filter
> The length of the time of wash
> The rise of sand during the washing period
> The turbidity at the end of 30 minutes operation
> The turbidity at the end of the run

The constants in operation were:

> The rate of wash
> The loss of head when washed
> The rate of filtration

Only variable conditions need enter into the discussion of the best grading of sand. A study of the plotted data and tables accompanying this review seem to indicate that the only basis of judgment for making a selection is the length of filter run. Filter Nos. 1, 3 and 7 exhibit longer runs than do any of the other filters. This fact is deduced from the graphs and is substantiated in table 1 which gives

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[1] Chief Chemical Engineer, Chain of Rocks Plant, St. Louis, Mo.

the average run in hours and minutes of all filters.   Mr. Armstrong, in a letter to our laboratory, dated September 27, 1929, mentions observing this fact early in the test.   He remarks also "It was also interesting to note that the cities of Chicago and Denver showed similar results, although the length of filter runs were much shorter than yours, due probably to the high organic content of the water."

The length of the time of wash did not vary a great deal for any of the experimental units and was only about half as long as the time required to wash the plant unit No. 8.   These averages are also given in table 1.

The rise of sand in inches during washing could not legitimately be considered as having a bearing on the selection of the best grading and depth of sand for the reason that it was used as an index to

TABLE 1

EXPERIMENTAL FILTER NUMBER	AVERAGE RUN	AVERAGE TIME OF WASH	AVERAGE TURBIDITY	
			After 30 minutes	At end of run
	hrs.-min.	minutes	p.p.m.	p.p.m.
1	80–30	2.60	0.88	0.06
2	50–30	2.27	0.82	0.09
3	72–54	2.35	0.89	0.06
4	37–06	2.16	0.82	0.06
5	43–00	2.18	0.81	0.07
6	43–00	2.27	0.74	0.08
7	71–06	2.33	0.78	0.07
8	57–18	4.16	0.36	0.11

control the rate of washing.   It should therefore really be considered as one of the constants of the test.   As noted in the original instructions for the operation of these units, it was believed that the rise of sand during washing would vary considerably as the sand became coated with mud and organic matter.   As the turbidity of the applied water varied between the limits of a maximum of 16, a minimum of 3, with an average of 7, the sand evidently did not become badly coated and presents a clean appearance in the tubes, even to this day. Apparently it would take a number of years to determine just what effect a coating on the sand would have on the amount of wash water used.   The rate of wash was not varied as recommended in the original instructions, but was held at a 3 foot rise per minute for the experimental units and a 2.083 foot rise for the plant unit No. 8.

Filter No.1

No.1 Filter Run in Hours
No.2 Turbidity after 30 Min.
No.3 Turbidity at End of Run

Filter No.2

Filter No.3

Filter No.4

Turbidity - Parts per Million

Filter Run - Hours

Test Number
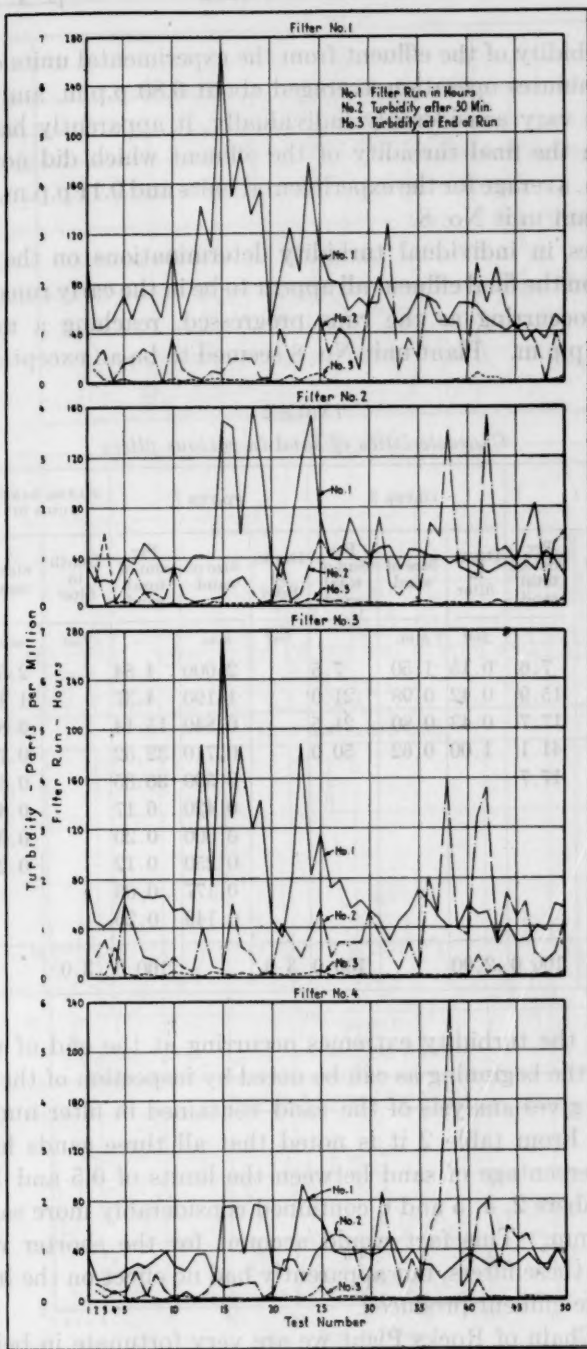
The turbidity of the effluent from the experimental units collected after 30 minutes operation averaged about 0.80 p.p.m. and while it seemed to vary considerably individually, it apparently had no influence on the final turbidity of the effluent which did not exceed 0.09 p.p.m. average for the experimental units and 0.11 p.p.m. average for the plant unit No. 8.

Extremes in individual turbidity determinations on the samples collected on the final effluent all appear to be in the early runs, smaller extremes occurring as the runs progressed, reaching a maximum under 0.5 p.p.m. Plant unit No. 8 seemed to be an exception, how-

TABLE 2

Characteristics of sand in various filters

FILTER 1			FILTER 3			FILTER 7			FILTER SAND PREPARED FROM RIVER SAND		
Depth in filter	Size of sand	Per cent of total sand	Depth in filter	Size of sand	Per cent of total sand	Depth in filter	Size of sand	Per cent of total sand	Depth in filter	Size of sand	Per cent of total sand
feet	mm.		feet	mm.		feet	mm.		feet	mm.	
0.13	1.50	7.6	0.15	1.50	7.5		2.000	4.84		2.000	10.5
0.27	0.98	15.9	0.42	0.98	21.0		1.190	4.37		1.190	7.1
0.30	0.80	17.7	0.43	0.80	21.5		0.840	15.14		0.840	21.2
0.70	0.62	41.1	1.00	0.62	50.0		0.710	32.52		0.710	29.6
0.30	0.50	17.7					0.590	36.35		0.590	24.6
							0.420	6.17		0.420	5.9
							0.300	0.20		0.300	0.9
							0.250	0.12		0.250	0.2
							0.177	0.03			
							0.149	0.26			
1.70		100.0	2.00		100.0	3.0		100.0	3.0		100.0

ever, with the turbidity extremes occurring at the end of the run, instead of the beginning as can be noted by inspection of the graphs.

Table 2 gives analysis of the sand contained in filter numbers 1, 3 and 7. From table 2 it is noted that all three sands have the greatest percentage of sand between the limits of 0.5 and 1.0 mm. Filter numbers 2, 4, 5 and 6 contained considerably more sand finer than 0.5 mm. This fact would account for the shorter runs exhibited by these filters, but apparently had no effect on the improvement of the effluent produced.

At the Chain of Rocks Plant we are very fortunate in being able

Filter No. 5

No.1 Filter Run in Hours
No.2 Turbidity after 30 Min
No.3 Turbidity at End of Run

Filter No. 6

Filter No. 7 (Control Unit)

Filter No. 8 (Large Filter No. 21)

Turbidity - Parts per Million
Filter Run - Hours

Test Number

1315
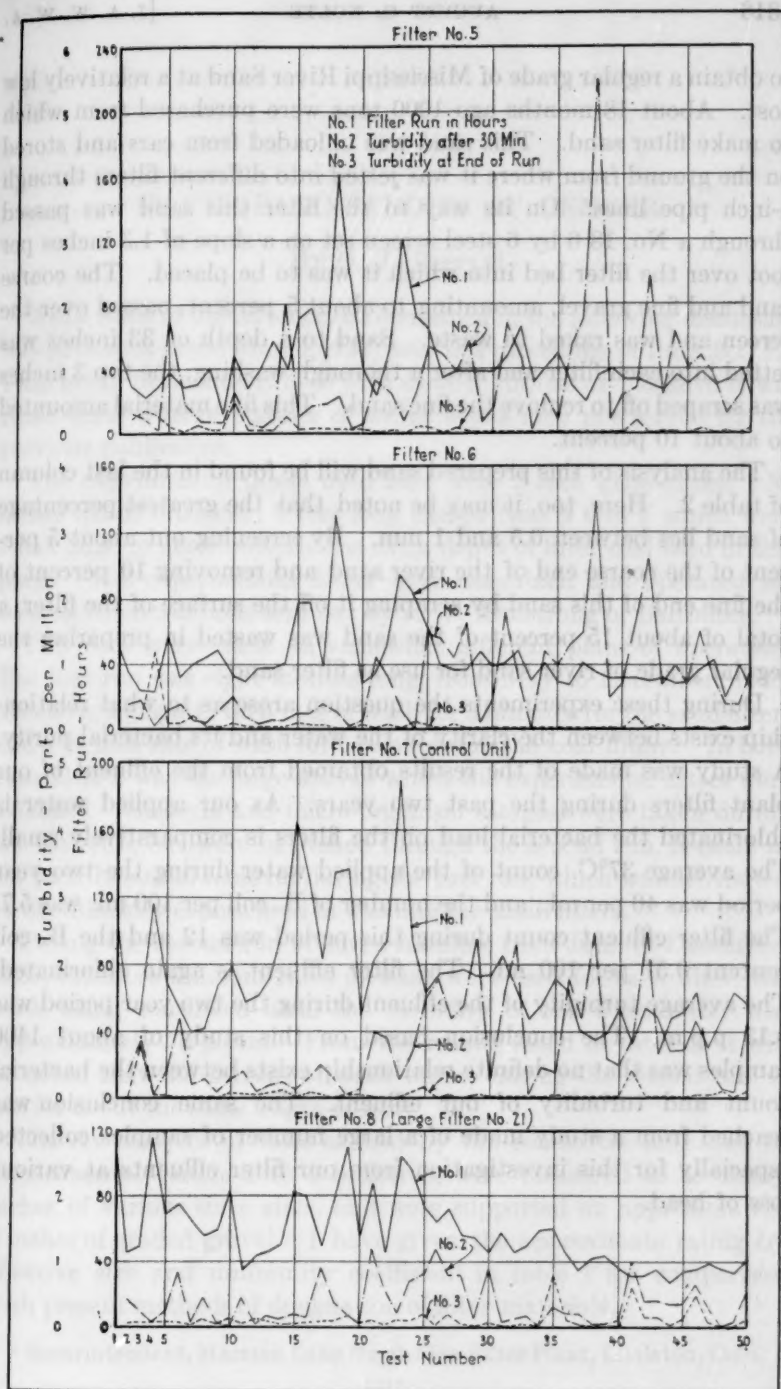
to obtain a regular grade of Mississippi River Sand at a relatively low cost. About 18 months ago 1000 tons were purchased from which to make filter sand. This sand was unloaded from cars and stored on the ground from where it was jetted into different filters through 4-inch pipe lines. On its way to the filter this sand was passed through a No. 18 6 by 6 steel screen set on a slope of 1.3 inches per foot over the filter bed into which it was to be placed. The coarse sand and fine gravel, amounting to about 5 percent, passed over the screen and was raked to waste. Sand to a depth of 33 inches was jetted into each filter and after a thorough washing, the top 3 inches was scraped off to remove the fine sand. This fine material amounted to about 10 percent.

The analysis of this prepared sand will be found in the last column of table 2. Here, too, it may be noted that the greatest percentage of sand lies between 0.5 and 1 mm. By screening out about 5 percent of the coarse end of the river sand and removing 10 percent of the fine end of this sand by scraping it off the surface of the filter, a total of about 15 percent of the sand was wasted in preparing the regular grade of river sand for use as filter sand.

During these experiments the question arose as to what relationship exists between the clarity of the water and its bacterial purity. A study was made of the results obtained from the effluent of our plant filters during the past two years. As our applied water is chlorinated the bacterial load on the filters is comparatively small. The average 37°C. count of the applied water during the two year period was 40 per ml., and the number of B. coli per 100 ml. was 5.7. The filter effluent count during this period was 12 and the B. coli content 0.57 per 100 ml. The filter effluent is again chlorinated. The average turbidity of the effluent during the two year period was 0.13 p.p.m. The conclusion based on this study of about 1400 samples was that no definite relationship exists between the bacterial count and turbidity of our effluent. The same conclusion was reached from a study made of a large number of samples collected especially for this investigation from our filter effluents at various loss of head.

FILTER SAND STUDIES AT DENVER

By O. J. Ripple[1]

While some of the information contained in the following discussion was published in the March, 1931, issue of Proceedings of the A.S.C.E., I am going to include it here with some additions as I believe it will reach quite a large number of water works men not served by the previous publication.

The experimental filters used here were 2 inch glass tubes, 1.88 inches inside diameter, 5 feet long except Nos. 9 and 7 which were 7 feet long, to accommodate the extra depth of filter coal used in this plant. The experimental filters were assembled and operated according to instructions sent out by J. W. Armstrong of Baltimore.

Two runs were made on somewhat different plans of operation. The first run was eight months long, from May to December, 1928, inclusive, using a 40 inch rate of wash for 3 minutes for the sand filters and a 24 inch rate of wash for 5 minutes for the coal filters. In both runs filter 8 was the large unit to which the experimental filters were attached. Bacteria and micro-organism samples were taken during the first run, but were not taken through the second run, because of the poor bacterial removal during the first run, which was attributed to the difficulty experienced in washing the small filters.

During the second run of seven months, from June to December, 1929, inclusive different rates of wash were used for each filter, the wash being continued until the turbidity of the wash water was approximately 70 p.p.m. From the results listed in table 2, it will be noted that the percent of wash water, during the second run, is less for about the same length of runs.

The six samples of sand as sent by the committee and used in Experimental Filters 1 to 6 inclusive, were measured as so many inches of various sieve sizes, and were supported on approximately 6 inches of graded gravel. I have given the approximate rating by effective size and uniformity coefficient in table 1 for comparison with present methods of designation of filter materials.

[1] Superintendent, Marston Lake North Side Filter Plant, Littleton, Colo.

1317

TABLE 1

Characteristics of experimental filters at Denver, Colo.

FILTER NUMBER	DEPTH, INCHES	MATERIAL	EFFECTIVE SIZE, MM.	60 PERCENT SIZE, MM.	UNIFORMITY COEFFICIENT
1	24	Sand	0.36	0.64	1.76
2	24	Sand	0.32	0.53	1.67
3	24	Sand	0.61	0.75	1.23
4	30	Sand	0.34	0.62	1.82
5	24	Sand	0.40	0.58	1.45
6	20	Sand	0.41	0.57	1.39
7	47	Coal	0.55	1.10	2.00
8	47	Coal	0.56	1.18	2.05
9	24	Coal top	0.83		1.10
	15	Sand bottom	0.50		1.30

TABLE 2

Data obtained from two runs, experimental filters at Denver, Colo.

FILTER NUMBER	NUMBER OF RUNS	MATERIAL	AVERAGE LENGTH RUN IN HOURS	PERCENT WASH WATER	RATE OF WASH, INCHES PER MINUTE	RISE SAND, INCHES	EFFLUENT TURBIDITY, P.P.M.
First run							
1	165	Sand	17.7	4.55	40.0	13.1	0.549
2	196	Sand	7.1	10.79	40.0	24.5	0.507
3	139	Sand	24.8	3.16	40.0	11.7	0.541
4	190	Sand	12.1	7.20	40.0	20.5	0.517
5	190	Sand	11.2	7.86	40.0	24.6	0.523
6	188	Sand	11.9	6.95	40.0	17.5	0.516
7	109	Coal	33.5	2.44	24.0	20.2	0.474
8	183	Coal	26.3	2.47	18.0	14.5	0.357
Second run							
1	134	Sand	17.4	2.48	50.6	23.4	0.431
2	134	Sand	8.1	4.74	36.7	23.9	0.414
3	128	Sand	23.6	1.96	55.2	21.9	0.397
4	134	Sand	11.9	2.71	31.3	18.1	0.419
5	134	Sand	10.6	3.58	37.9	24.4	0.433
6	134	Sand	10.8	3.47	43.3	25.2	0.430
7	113	Coal	29.6	1.67	28.4	25.9	0.385
8	126	Coal	35.9	1.88	17.7	14.3	0.350
9	67	Coal and sand	50.4	1.28	31.4	36.7	0.359

Filter 9 was a combination of sand and coal and was not run during the first period of operation.

From table 2 it will be noted that the coarsest sand filter (No. 3) gave the best results of any of the sand filters, number 9, a combination of sand and coal, gave better results than either the sand or coal used separately, and it is my opinion that this type of filter material can be used in many instances at a material saving in wash water.

TABLE 3

Analysis of raw water, during above two runs

	FIRST RUN			SECOND RUN		
	Maximum	Mini-mum	Average	Maximum	Mini-mum	Average
Turbidity, p.p.m........	8.3	4.9	6.7	12.6	3.2	7.2
Temperature, degrees Cent..................	19.8	3.2	10.4	20.4	3.0	10.2
Alkalinity, p.p.m.:						
pH....................	8.42	7.85	8.22	8.11	7.78	7.96
MO..................	92	67	78	78	71	75
Phenol...............	5	1	3	1	0	0.5
Bacteria per cubic centimeter..................	521	208	334	585	53	264
Micro-organisms, Std. U. per cubic centimeter....	1,757	305	656	1,651	344	675
Chemicals, g.p.g:						
Alum..................	0.667	0.123	0.254	0.443	0.168	0.256
Dry sodium aluminate..	0.133	0.075	0.113	0.114	0.049	0.083

The raw water treated during the two runs was characterized by a low turbidity, as shown in table 3. This is due to its storage in Marston Lake, which has a capacity of 6,500,000,000 gallons.

THE CHEMISTRY OF CHLORAMINES

By J. F. T. Berliner[1]

In the term "chloramine" the chemist includes all amino ($-NH_2$) or imino ($=NH$) groups in which the hydrogen has all or in part been replaced by chlorine. The term "chloramine" to those in the medical and pharmaceutical professions refers more or less specifically to the mono- and di-chlorine substituted toluene sulphonamide derivatives, usually known as Chloramine-T, Halazone or Dichloramine-T. To those interested in the treatment of water and related endeavors the term is restricted to the chlorine substitution products of ammonia. Even in this field the term "chloramine" has come, through general usage, to indicate monochloramine, NH_2Cl. However, in almost every application of chloramines to water treatment we are dealing with a variable mixture of the mono- and di-chloramine ($NHCl_2$). Under certain extreme conditions the trichloramine or nitrogen trichloride (NCl_3) can be formed in the treatment of water, but it is seldom encountered and is decidedly undesirable.

Chloramines are produced in water by the introduction of ammonia and chlorine. The process is not new. It was first proposed about 1904. Interest in its possibilities has been dormant for more than a score of years. In fact, widespread interest and application of the ammonia-chlorine treatment to municipal and private water supplies began just about one year ago. On January 1, 1930, there were but about five installations in the United States employing this process. By September 1, 1930, this number had increased to 70 and on May 25, 1931, the recorded number of installations is 190.

This remarkable and sudden development has centered considerable interest on the basic chemistry of the process. It has, therefore, been considered timely to summarize briefly the available information on the nature and properties of chloramines, their reactions and interrelations, and to assemble for reference citations to the more important studies on these phases.

The existence of chloramines had been noted as early as 1811 by

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P. L. Dulong (1), who first recorded the preparation of the explosive trichloramine and, as Mellor (2) states, continued to work on it after it had caused him the loss of three fingers and one eye.

A voluminous amount of investigation and study has been reported on the properties and reactions of trichloramine, and of the three chloramines this is by far the most completely known. Unfortunately this chloramine is of the least significance or interest to the members of the water works profession. Because of its relative unimportance little space will be devoted to it. However, it may be of value to note that two excellent summaries of the literature on the properties and preparation of trichloramine exist, one supplied by Mellor (2) (see ref. No. 1, page 611) and the other by Gmelin (3). The latter is somewhat more complete.

Very little has been published on the physical and chemical properties of the pure mono- and di-chloramines. In fact, practically nothing is known of the properties of dichloramine except that as regards odor, volatility, partial pressure in solution, etc.; they are intermediate between those of mono- and tri-chloramine.

It may be of interest to first examine the physical and chemical properties of the pure chloramines and then to consider their reactions, properties and interrelation in aqueous solutions.

PHYSICAL PROPERTIES AND PREPARATION OF PURE CHLORAMINES

The chloramines when pure are all rather oily, extremely unstable liquids with an acrid odor. They are all water-soluble. The trichloramine is, however, but slightly soluble in water. The solubilities of the three chloramines in various organic solvents vary considerably. This property affords a means for their separation. For instance, monochloramine is practically insoluble in carbon tetrachloride, the dichloramine is soluble to a small extent and the trichloramine is very soluble in this solvent. Monochloramine is but slightly soluble in ethyl ether but the dichloramine is highly soluble in this material.

The three chloramines may be formed under varying conditions through reaction between chlorine or hypochlorous acid or its derivatives and ammonia or certain ammonium salts. The usual method of obtaining the pure chloramines is to first prepare them in the form of a strong aqueous solution and then extract these solutions with suitable organic solvents or to distil the chloramine from the aqueous solution by vacuum treatment or gas washing.

Monochloramine

Because of the ease of decomposition in its pure free state, little is known of the physical properties of monochloramine.

It is an oily colorless liquid, which on cooling solidifies to an amorphous glassy mass and under certain conditions, in part takes on a white crystalline form. It has, when concentrated a somewhat acrid pungent odor (24, 26, 27) lying between that of ammonia and trichloramine (4). The melting point of monochloramine is reported as −66°C. If ammonia is present as an impurity, the melting point is reduced 3 or 4°C. At −65°C. the liquid has a very low vapor pressure. However, at about −50°C. violent and at times explosive decomposition occurs. This decomposition yields nitrogen, chlorine, and a voluminous mass of yellow oily droplets of trichloramine interspersed with white crystalline ammonium chloride (4).

Monochloramine is about as soluble in water as ammonia. It is but slightly soluble in benzol, chloroform and carbon tetrachloride, though readily soluble in ethanol and ether. The partition ratio between water and ether with a fresh solution (4) is about 1:0.97. On standing, some monochloramine is converted to trichloramine and this is completely absorbed from the ether layer. The partition ratio of monochloramine between water and chloroform is 1:0.25 (5).

When a concentrated aqueous solution of monochloramine is maintained at a low temperature it deposits yellow crystals of chlorine hydrate. At 0°C. decomposition with formation of nitrogen soon commences (4). The decomposition increases at an accelerated rate because of the catalytic action of the liberated acid. The presence of a very small amount of ammonia greatly stabilizes (4) the aqueous solution and there is evidence that one of the fundamental requirements for the formation of chloramine is a decided excess of ammonium ions (30).

Monochloramine is prepared by washing a relatively concentrated chloramine solution with benzol, chloroform or carbon tetrachloride to remove the tri- and di-chloramine. A slight excess of ammonia is then added to stabilize the monochloramine and the solution distilled in a vacuum employing a drying agent such as soda-lime and rapidly condensing the resultant vapors with liquid air or similar refrigerants. The excess ammonia is removed by vacuum treatment of the refrigerated condensate (4).

Dichloramine

Practically nothing is known regarding the physical properties of dichloramine, except that, as stated before, they are intermediate between those of mono- and tri-chloramine. Its existence was probably first noted by Gatterman (8).

The following partition ratios for dichloramine in water have been reported; to chloroform 1:1.88; to carbon tetrachloride 1:0.85; to ether 1:47.3 (5). Dichloramine may be obtained by preparing a chloramine solution under such conditions of acidity that only the dichloramine and some trichloramine may be present. The trichloramine is removed as in the preparation of monochloramine. The dichloramine can then be concentrated by extraction with ethyl ether.

Trichloramine

The physical properties of trichloramine or nitrogen trichloride have been determined to some degree of completeness. Its boiling point is reported as $+71°C$. It is still liquid at $-40°C$. Its density is about 1.6. Trichloramine is only slightly soluble in water but readily soluble in benzol, carbon bisulphide, chloroform, ether, ethanol, carbon tetra-chloride, sulphur dichloride and phosphorus trichloride.

Trichloramine is a yellow, thick oily liquid with a pungent odor resembling chlorine. The liquid is stated to be a non-conductor of electricity (7). It is an explosive compound which detonates spontaneously, especially in the presence of organic matter. It explodes with a bright flash and a loud report and has a violent detonating wave. It is very liable to spontaneous decomposition and extreme care should be employed in handling. It can be detonated by exposure to magnesium light or direct sunlight (8) and mere contact with many substances causes rapid decomposition.

Trichloramine can be directly formed through mixing together the proper proportion of anhydrous solutions of ammonia and chlorine in chloroform or pentane cooled to $-90°C$. or lower ($4NH_3 + 3Cl_2 = 3NH_4Cl + NCl_3$). The by-product, ammonium chloride, precipitates out and the trichloramine can be separated from the solvent by vacuum distillation. Trichloramine may also be formed by passing air or other gas through an aqueous solution containing trichloramine and absorbing the trichloramine carried over by the gas in a suitable solvent. A number of interesting methods for the preparation

of the trichloramine have been devised and are well summarized in the treatises of Gmelin (3) and of Mellor (2).

CHEMICAL PROPERTIES OF THE CHLORAMINES

The preparation of the aqueous solutions of the three chloramines involves two essentials, namely: The proper proportions of ammonia to chlorine and the proper control of the acidity or hydrogen-ion concentration of the solution. Before considering this phase of the subject, the general chemical properties of the chloramines should be considered. Monochloramine is the most important from our viewpoint and will be considered first and in most detail.

Monochloramine

It is well known that when aniline is treated with chlorine or hypochlorite solution, a blue-violet coloration is obtained. However, if ammonia is present prior to the addition of the chlorine, no coloration is produced (23). Similarly, the presence of ammonia prevents the usual reaction between phenol and chlorine. Mixing concentrated solutions of ammonia and of sodium hypochlorite causes an immediate evolution of nitrogen and the formation of some nitrates and other nitrogen oxidation products. If the solutions are diluted before mixing, nitrogen will only be evolved, if at all (15), after a considerable period of time. Departure from the relation $2NH_3 + 3Cl_2 = HCl + N_2$ appears to increase with dilution. These and other factors led investigators to the conclusion that compounds intermediate between ammonia and nitrogen were being formed. These compounds are the chloramines and under general conditions predominantly monochloramine.

The formation of monochloramine through the action of chlorine, hypochlorous acid or a hypochlorite on ammonia is considered to be preceded by the intermediate formation of ammonium hypochlorite,

$$Cl_2 + H_2O = HOCl + HCl$$
$$HOCl + NH_3 = NH_4OCl$$
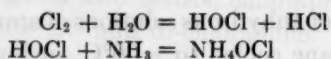
Ammonium hypochlorite is very unstable and is rapidly transformed into monochloramine and water (28)

$$NH_4OCl = NH_2Cl + H_2O$$
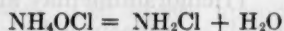
In this reaction some traces of nitrous oxide have been evidenced (32, 6) in relatively concentrated solution. The formation of

nitrites and nitrates through oxidative action on dilute chloramine solutions has also been noted (2, 33, 6).

Chapin has concluded (6) that elementary chlorine is inert towards ammonium ions, requiring conversion to hypochlorous acid before the reaction occurs.

Braidech (33) has recently made the interesting suggestion that a buffering or stabilizing action may result from the formation of complex molecular aggregates of the hydrates of ammonia and chlorine, in equilibrium with the associated molecules which compose the intramolecular structure of water at ordinary temperatures.

Monochloramine reacts neutral in water. In aqueous solution it tends to decompose at a rate commensurate with the concentration (under equivalent temperature conditions) into nitrogen, ammonium chloride and hydrochloric acid (10),

$$3NH_2Cl = N_2 + NH_4Cl + 2HCl$$
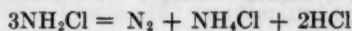
This reaction is rapidly accelerated by catalysts such as sponge platinum (4) or acids (33).

In very concentrated aqueous solutions (of the order of about 10 per cent, the presence of trichloramine has been noted (4) in its decomposition products.

Raschig (13) has recorded the decomposition reaction for concentrated monochloramine solutions to be

$$4NH_2Cl = 2NH_4Cl + N_2 + Cl_2$$

The tendency has been reported for the formation of hypochlorous acid through the hydrolytic reaction

$$NH_2Cl + H_2O = NH_3 + HClO$$

which may occur under certain conditions (24).

Monochloramine is decomposed by concentrated hydrochloric acid according to the reaction (31)

$$NH_2Cl + 2HCl = NH_4Cl + Cl_2$$
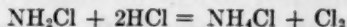
The reaction with hydriodic acid is similar (10). Diluted hydrochloric acid yields condition for the formation, through successive stages, of trichloramine,

$$3NH_2Cl + 2HCl = NCl_3 + 2NH_4Cl$$
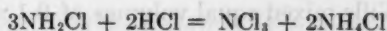
The acidification of monochloramine involves the formation of dichloramine which is considered to progress in two steps (6),

$$NH_2Cl + HCl + H_2O = NH_4Cl + HClO$$
$$NH_2Cl + HClO = NHCl_2 + H_2O$$
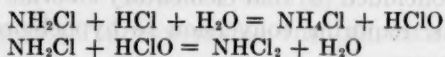
The conversion of di- to tri-chloramine probably involves analogous steps.

It has been considered more or less typical for the chloramines to decompose or rearrange themselves in such a manner as to re-form ammonia or ammonium salts. There is the opposing tendency for the more complete decomposition to elemental nitrogen. Both of these reactions manifest themselves in the alkaline decomposition of monochloramine (12),

$$3NH_2Cl + 3KOH = NH_3 + N_2 + 3KCl + 3H_2O$$

or (13)

$$3NH_2Cl + 2NaOH = NH_4Cl + N_2 + 2NaCl + 2H_2O$$
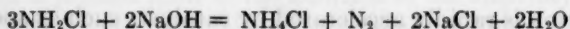
More recent considerations (6), however, indicate that the alkaline treatment of monochloramine may lead to the formation of hypochlorites.

When an attempt is made to prepare concentrated chloramine solutions by treating ammonia solutions with sodium or calcium hypochlorite, chloramine is formed in the presence of relatively high concentrations of alkali ($NH_3 + NaOCl = NH_2Cl + NaOH$). The chloramine that is formed tends to be immediately decomposed by the alkali to yield nitrogen and ammonia according to the above reactions. The final result of the reaction between concentrated ammonia and an alkaline hypochlorite solution is, therefore,

$$2NH_3 + 3NaOCl = 3NaCl + 3H_2O + N_2$$
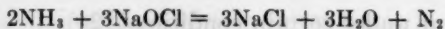
There is also the contemporary side reaction involving the intermediate formation of ammonia and alkali to form hydrazine (13),

$$NH_2Cl + NH_3 + NaOH = N_2H_4 + NaCl + H_2O$$
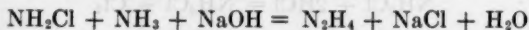
Another side reaction that ensues is the formation of chlorate,

$$3NH_2Cl + 3NaOH = 3NH_3 + KCl + KClO_3$$
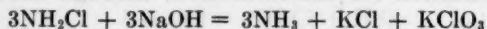
Marckwald and Wille mixed equal volumes of 0.1 N monochloramine solution and 0.25 N potassium hydroxide solution and noted the for-

mation of nitrogen according to the reactions previously given. When the reaction had been completed through warming, about 12.5 percent of the chlorine in the original chloramine was found combined in the form of chlorate.

Another side reaction has been noted by Thiele (15) which indicates the formation of hydroxylamine (NH_2OH). This involves further reaction between hydroxylamine and hydrazine to form ammonia, nitrogen and water. The presence of hydrazoic acid (N_3H) has also been noted in the products of the alkaline decomposition of monochloramine (22).

Chloramine reacts towards ammonia essentially as with the alkali hydroxides, thus:

$$3NH_2Cl + 2NH_3 = N_2 + 3NH_4Cl$$
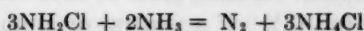
In studying this reaction Thiele (15) was the first to note the presence of a small amount of a strong reducing substance. Although he did not identify the material it was later recognized to be hydrazine,

$$NH_2Cl + NH_3 = N_2H_4 + HCl \quad (22)$$
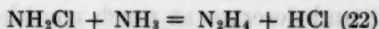
This method for the production of hydrazine has been the subject of some investigation and it has been found that substances that increase the viscosity of a solution, such as glycerine, sugar, albumin, casein, etc., act as catalysts towards the formation of hydrazine. Raschig (12, 13, 14) claims an 80 percent conversion.

It should here be emphasized that the above reactions refer almost entirely to relatively concentrated solutions. In somewhat dilute solutions, ammonia acts as a powerful stabilizer for chloramine and prevents its decomposition or reversion to nitrogen, ammonium chloride, ammonia, hydrazine, hydrochloric or hypochlorous acid, hydroxyl amine, chlorates, hydrazoic acid, chlorine or the other chloramines. Dilute chloramine solutions in the presence of an excess of ammonia may be distilled without decomposition (4).

The formation of hydrazine through the reaction between ammonia and monochloramine is not of importance in the treatment of water, as there is no evidence that this reaction will take place in dilute solution of chloramine. Solutions containing about 150 parts per million chlorine and 250 percent excess ammonia will not develop a detectable trace of hydrazine.

There are a number of other reactions of interest but only those of

more immediate importance have been considered. Chloramine reacts with hypochlorite to form trichloramine (4),

$$NH_2Cl + 3NaOCl = NCl_3 + 2NaOH$$
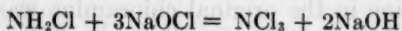
Sulphites or bisulphites reduce chloramine solutions (1)

$$NH_2Cl + SO_2 + H_2O = NH_4 \cdot HSO_4 + HCl$$
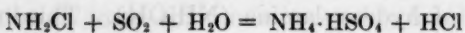
However, a series of side reactions involve the formation of amido sulphonic acid derivatives (13),

$$NH_2Cl + Na_2SO_3 = NH_2SO_3Na + NaCl$$
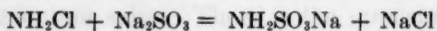
When a solution of monochloramine is treated with a solution of a silver salt, the solution remains clear at first then gradually silver chloride is precipitated (3). If a neutral or ammoniacal solution of silver salt be employed, metallic silver will be deposited slowly (29, 13).

Dichloramine

Dichloramine is a close second to monochloramine in importance in water treatment and deserves as much consideration. Unfortunately very little is known of the chemical properties of this substance. However, it may be judged that in many respects its behavior will be analogous to that of monochloramine.

Like monochloramine it will liberate iodine in a starch-iodide solution. On treatment with concentrated ammonia solution, decomposition ensues according to the reaction (13)

$$3NHCl_2 + 7NH_3 = NH_4Cl + 2N_2$$
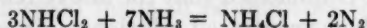
The acidification of dichloramine leads to the formation of trichloramine while the treatment of a dilute solution of dichloramine with alkali forms some monochloramine and involves a number of reactions, the principal one appearing to be the formation of hypochlorites (6) thus

$$NHCl_2 + 4NaOH = 3NaCl + NaOCl + 3H_2O + N_2$$
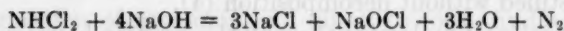
The presence of oxidation products of nitrogen such as nitrites also have been noted.

Trichloramine

The chemical properties of trichloramine, the chloramine of least interest in water treatment, are better known than those of the other chloramines.

Trichloramine in non-aqueous solutions decomposes, at a rate depending on the temperature and concentration, to nitrogen and chlorine (16),

$$2NCl_3 = N_2 + 3Cl_2$$
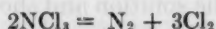
In aqueous solutions this reaction may also take place to a small extent, but the principal reaction appears to be, especially under acid conditions, the formation of hypochlorous acid according to the equation (17)

$$NCl_3 + 3H_2O = NH_3 + 3HClO$$

This reaction had led to the hypothesis that trichloramine is an amido anhydride of hypochlorous acid, since this acid and not hydrochloric acid is formed on hydrolysis (18). This view readily explains the ease of the reaction

$$NCl_3 + 4HCl = NH_4Cl + 3Cl_2$$

For further discussion of this point the works of Noyes (19) should be consulted.

Trichloramine reacts with an excess of ammonia to yield nitrogen or ammonium hypochlorite (24),

$$NCl_3 + 4NH_3 = NH_4Cl + N_2$$
$$NCl_3 + 2NH_3 + 3H_2O = 3NH_4OCl$$
$$NH_4OCl = NH_2Cl + H_2O$$
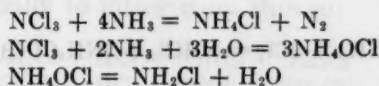
The treatment of nitrogen trichloride with alkali leads as with dichloramine to the formation of hypochlorites (6)

$$2NCl_3 + 6NaOH = 3NaCl + 3NaOCl + 3H_2O$$
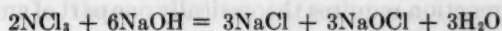
A number of side reactions are also involved, one of which results in the formation of nitrites.

Trichloramine in solution can be reduced to ammonia by such substances as sodium sulphite (21). Silver nitrate solutions deposit silver chloride on treatment with trichloramine (20).

It should be recorded that Porret (7) claims that below 0°C. no trichloramine could be formed and that temperatures above 32°C. are most favorable for its formation. He further claimed that sulphur, ammonium sulphide, powdered charcoal or dilution of the chlorine with 1/3 to 1/2 with air CO_2 or hydrogen inhibited the

formation of trichloramine. These claims have not been substantiated and to some extent have been refuted by subsequent evidence.

Bray and Dowell (21) concluded that the reaction depended primarily on the acidity of the solution and showed that in the presence of strong acids ammonia and chlorine reacted to form trichloramine, thus

$$NH_3 + 3Cl_2 = NCl_3 + 3HCl$$
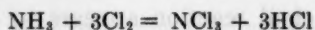
In strongly basic solutions nitrogen is formed, thus

$$8NH_3 + 3Cl_2 = N_2 + 6NH_4Cl$$

They considered these reactions to be a series of consecutive reactions (19), the first of which is the formation of monochloramine,

$$2NH_3 + Cl_2 = NH_2Cl + NH_4Cl$$
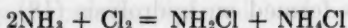
INTERRELATION OF CHLORAMINES

In the average treatment of water with ammonia and chlorine the two most predominant reactions between the ammonia and chlorine are

$$2NH_3 + Cl_2 = NH_2Cl + NH_4Cl \ (A)$$

and

$$3NH_3 + 2Cl_2 = NHCl_2 + 2NH_4Cl \ (B)$$
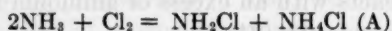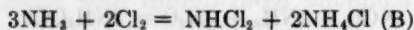
The relative proportion of mono- and di-chloramine formed, is governed principally by the final hydrogen ion concentration of the solution. Under average conditions the monochloramine reaction (A) predominates. This reaction requires theoretically one part of ammonia to two of chlorine (1:2). However, there is evidence indicating that an excess of ammonia or rather ammonium ions in solution is a requirement for chloramine formation (30, 5, 6). It has been established that the presence of an excess of ammonium ions, under proper conditions of hydrogen ion concentration, acts as a very active stabilizer in preventing the decomposition of monochloramine. The practical aspect of this is illustrated in the great number of installations that through numerous trials have found an optimum ratio of two parts of ammonia to three parts of chlorine (2:3).

The conditions which limit the formation of the three chloramines in aqueous solutions were not until very recently quite understood.

Bray and Dowell (21) state: "The reactions between chlorine and ammonia in aqueous solution furnish striking examples of changes in the nature of the product which result from variations in experimental conditions."

It has long been recognized in the reaction of ammonia and chlorine in solution that acidity leads to the formation of nitrogen trichloride and alkalinity towards monochloramine. Chapin has carefully studied the formation and transformation of the chloramines and has determined the ranges of hydrogen ion concentration limiting the existence of the three chloramines.

The concentration of hydrogen ions in the medium in which the reaction occurs is the only governing factor providing the reacting materials are rapidly mixed, sufficiently buffered and cooled, and there is an excess of ammonium ions (5). Chapin demonstrated that under these conditions, above a pH of 8.5, only monochloramine could be formed; in the range 4.4 to 5.0 dichloramine was formed and trichloramine could only be formed below pH of 4.4. Between the pH range 5.0 to 8.5 mono- and di-chloramine coexist in a ratio fixed by the hydrogen ion concentration of the solution. At neutrality (pH = 7) there are approximately equal amounts of mono- and di-chloramine present. Acidification of a monochloramine solution causes a gradual, proportional conversion to the dichloramine or trichloramine, or finally to nitrogen on storage.

$$2NH_2Cl + HCl = NHCl_2 + NH_4Cl$$
$$3NHCl_2 + HCl = 2NCl_3 + NH_4Cl$$
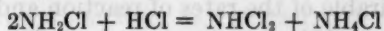
It should be pointed out that there is a factor of time involved in the conversion of one chloramine to another by control of the hydrogen ion concentration. These reactions are not completely reversible for when solutions of dichloramine and of trichloramine are treated with alkali there is a tendency for side reaction to occur, involving the loss of nitrogen and the formation of chlorides and hypochlorites. The extent of these side reactions is governed principally by the concentration of the reactants and the acidity and temperature of the solution.

From these considerations it is apparent that in the preparation of chloramine solutions it is essential that the acidity of the medium in which the reaction occurs be controlled before and during the progress of the reaction. In many instances attempts have been made and instructions have been published to prepare chloramine

solutions by passing ammonia into an aqueous solution of chlorine. If this chlorine solution is acid, as it may be unless sufficient alkali has been added or was originally present, different products than originally desired may be formed. For instance, the addition of ammonia to a solution of chlorine with an acidity of about pH 3 will at first lead to the formation of trichloramine and some products of side reactions. As the addition of the ammonia continues, the solution becomes progressively more alkaline and dichloramine and finally monochloramine are produced. The final concentration of chloramine will be considerably less than that which could be produced by the addition of the equivalent amount of chlorine to the same quantity of ammonia in solution.

In this inclusive subject of the chemistry of chloramines it is apparent that we are dealing with three compounds that can undergo numerous involved reactions and that are very closely interrelated. They exhibit unusual physical and chemical properties which are highly dependent on their state of concentration. A complete separate chemistry could be written for these compounds in various degrees of concentration. The aqueous solutions with a maximum concentration of about 0.0001 per cent (1.0 p.p.m.) are of most importance to those interested in water treatment. We have by allowable analogy a fair understanding of the chemistry of the chloramines at these concentrations. We do not have for these conditions as complete a knowledge as desirable of the rates of reaction and conversion, the effects of mass action of the reactants, or the influence of sulphates, carbonates and other such materials as are found or are added to water under treatment.

REFERENCES

(1) DULONG, Ann. Chim. Phys. (1), *86*, 37 (1813).
(2) MELLOR, A Comprehensive Treatise on Inorganic and Theoretical Chemistry, Vol. VIII, 1928, pp. 598–604.
(3) GMELIN, Handbuch der Anorganischen Chemie, 8th Ed., No. 6, p. 410–417 (1927).
(4) MARCKWALD AND WILLE, Ber., *56*, 1319–1325 (1923).
(5) CHAPIN, J.A.C.S., *51*, 2112–2117 (1929).
(6) CHAPIN, J.A.C.S., *53*, 912–920 (1931).
(7) PORRET, KIRK AND WILSON, Nicholson's Journal, *34*, 180, 276 (1813); Gilbert's Annalen, *47*, 56, 69 (1814).
(8) GATTERMAN, Ber., *21*, 751 (1888).
(9) MONFORT AND BARNES, Ill. Water Survey Bull., *16*, 218 (1918/19).

(10) RASCHIG, Verh. Ges deutsche Naturforsche u. Aertze, *2*, (1) 120 (1907);
    Ber., *40*, 4586 (1907); Z. ang. Chem., *19*, 1748, 1985, 2049, 2083 (1906);
    20, 2068 (1907).

(11) RASCHIG, Ber., *41*, 4194 (1908).

(12) RASCHIG, Chem. Ztg., *31*, 926 (1907).

(13) RASCHIG, Schwefel-und Stickstoff-Studien. Leipzig and Berlin (1924),
    pp. 50–78.

(14) RASCHIG, German Pat. No. 192,783 (1906); No. 198,307 (1908).

(15) THIELE, Lieb. Ann., *273*, 160 (1893).

(16) DAVY, Phil. Trans., *103*, 242, (1813).

(17) SELIVANOFF, Ber., *27*, 1012 (1894).

(18) SELIVANOFF, J. Russ Ges. Chem., *26*, 498 (1894); Ber., *25*, 3618 (1892).

(19) NOYES, J.A.C.S., *35*, 767 (1913); *39*, 912 (1917); *42*, 2173 (1920); Bull. Soc.
    Chim., (4) *35*, 425 (1924), and other papers.

(20) SERULLAS, Ann. Chim. Phys., (2) *42*, 209 (1829).

(21) BRAY AND DOWELL, J.A.C.S., *39*, 896 (1917).

(22) STOCK, Ber., *53*, 837 (1920).

(23) TOLLENS, Ber., *2*, 189 (1869); Beilstein (2nd Ser.), *11B*, 442.

(24) CROSS AND BEVANS, Proc. Chem. Soc., *6*, 22 (1890); Ber., *24*, 689 (1891).

(25) CROSS, BEVANS AND BRIGGS, J. Soc. Chem. Ind., *27*, 269 (1908); Chem. Ztg.,
    32, 369 (1908).

(26) LUNGE AND SCHOCH, Ber., *20*, 1474 (1887).

(27) ARMSTRONG, Proc. Chem. Soc., *6*, 24 (1890).

(28) SCHÖNBEIN, J. Prakt. Chem., *84*, 385 (1861).

(29) LUTHER AND SCHILOW, Ztsch. Phys. Chem., *46*, 818 (1903).

(30) TILLEY AND CHAPIN, Journ. Bact., *29*, 295 (1930).

(31) BODENSTEIN, Z. phys. Chem., Abt. A, *137*, 131 (1928).

(32) RAYLEIGH, Proc. Roy. Soc. (London), *64*, 95 (1898).

(33) BRAIDECH, Water Works Eng., *84*, 433 (1931).

THE BACTERICIDAL EFFICIENCY OF THE AMMONIA-CHLORINE TREATMENT

By H. H. Gerstein[1]

The marked success which the ammonia-chlorine treatment has met during the past two years throughout the United States in the prevention of tastes and odors commonly associated with the chlorination of water supplies, coupled with the simplicity and the low operating and first costs of the process, has made it expedient for Chicago to give the treatment careful consideration.

It was apparent from the beginning that, in giving consideration to the use of the ammonia-chlorine treatment, it would be necessary to determine carefully what the bactericidal velocity of the treatment was as compared with that obtained in ordinary chlorination practice. A slowing up of the bactericidal action would be of great disadvantage if the treatment were to be applied at the Chicago pumping stations, because of the lack of protection it would afford to nearby consumers, many of whom receive their water in less than three minutes after it is chlorinated. If such delayed action actually occurred, and it was considered advisable to use the treatment, it would be necessary to provide sufficient detention period to obtain the required bactericidal effect before the water was pumped into the system.

The literature on this subject was not conclusive; some observers noticed a decided lag in bactericidal velocity, while others observed none. The investigations made by Holwerda[2] and more recently by Baker and Schmelkes[3] indicate that the widely varying effects observed were probably due to the application of the treatment to waters of different hydrogen-ion concentrations.

Extensive research work was begun in September, 1930, under the direct supervision of Physical Chemist John R. Baylis, and in collaboration with the writer. The technicians on the work were Senior Bacteriologist John H. Mills and Senior Chemist Oscar Gullans of Mr. Baylis' staff, and Junior Bacteriologist Catherine Weir and Senior Chemist Armada T. Weathers from the Health Department Water

[1] Sanitary Engineer, Department of Public Works, Chicago, Ill.

[2] Holwerda, Meded, Dienst Volksgezondheit Ned. Ind. 17: 251, 1928.

[3] Baker and Schmelkes, Research Publication 305, Wallace and Tiernan Company, 1931.

Analysis Laboratory, who were detailed to the work through the courtesy of Director of the Health Department Laboratories, Dr. F. O. Tonney.

Credit is due to Mr. Baylis and the technicians working under him who were responsible for the carrying out of this work, and to the officials of the Department of Public Works who provided all the necessary facilities. Acknowledgment is also made of the coöperation received from Dr. Tonney and his staff.

It would be impracticable to present the results of all the studies made in connection with the research work. The writer will therefore confine himself to such data as concern the bactericidal efficiency of the treatment and related matters.

TECHNIQUE USED IN BACTERICIDAL EXPERIMENTS

The water to be tested was prepared by seeding a pure culture of Bacterium coli in 5-gallon portions of raw lake water taken from the Dunne Crib tunnel system. The organisms used were of a typical methyl red positive, Voges-Proskaur negative strain. The growth of one agar slant culture was suspended in 100 cc. of distilled water and 10 cc. of this suspension added to the 5 gallons of raw water. After adding the organisms, agar plate dilutions were prepared and incubated for 18 hours at 37°C. to determine the number of B. coli per cubic centimeter in the water. After determining the number of organisms, the B. coli content was adjusted by dilution with enough raw water to obtain a concentration of approximately one million per 100 cc.

The raw water was then distributed into two liter glass stoppered bottles, the required amount of ammonia added to each bottle and then the contents thoroughly mixed by shaking. The ammonia was permitted to remain in contact with the water for thirty minutes, after which time the chlorine was added to the bottles. After the addition of the chlorine the contents of bottles were shaken thoroughly before the 5-minute contact samples were withdrawn, and also shaken before each of the following samples were withdrawn.

The two liter bottles were prepared for the test by disinfecting with a chlorine solution of approximately 5.0 p.p.m. concentration. This solution was allowed to remain in the bottle for 18 hours, after which time the bottles were filled with filtered water containing no chlorine. Before being used the bottles were thoroughly rinsed with the raw water with which they were later filled.

The ammonia was applied in the form of ammonium hydroxide solution having approximately 0.1 mgm. of NH_3 to each cubic centimeter. During the first few months of the work the solutions were checked each day by Nesslerization. The consistent results, however, indicated that the solutions were quite stable and thereafter the checks were made at less frequent intervals.

The chlorine solution was prepared the day previous to its use by diluting a highly chlorinated stock solution obtained from the discharge of a solution feed chlorinator with double distilled water to a concentration of approximately 0.2 mgm. of chlorine per cubic centimeter. This solution was stored in the refrigerator and was checked just before use with N/100 sodium thiosulphate. The sodium thiosulphate solution was checked every two weeks with N/100 potassium dichromate.

TABLE 1

Removal of residual chlorine and chloramine by peptone solution

CC. OF $\frac{5}{8}$ PEPTONE PER 100 CC. H_2O	NH_3 APPLIED, P.P.M.	Cl_2 APPLIED, P.P.M.	RESIDUAL Cl_2, P.P.M. TEMPERATURE, 22.5°C.		RESIDUAL Cl_2, P.P.M. TEMPERATURE, 1.3°-5°C.	
			After 5 seconds	After 5 minutes	After 5 seconds	After 5 minutes
1	0	0.5	0	0	0	0
1	0.3	0.5	0	0	0	0
2	0.3	0.5	0	0	0	0
4	0.3	0.5	0	0	0	0
0	0	0.5	0.40*	0.42	0.27	0.12
0	0.3	0.5	0.48	0.48	0.45	0.45

* After 20 seconds.

The bactericidal efficiency was determined by examination of samples withdrawn by means of a sterile 100 cc. pipette, at intervals of 5 minutes, 30 minutes, 2 hours and 24 hours after the chlorine had been added. The samples were then immediately placed in sterile bottles, which contained 1 cc. of a 5 percent solution of Bacto-peptone. The peptone solution was used for the purpose of stopping bactericidal action by removal of the residual chlorine. A series of tests shown in table 1 indicate that this concentration of peptone was capable of completely removing the residual chlorine, as indicated by the ortho-tolidin test, in less than 5 seconds. In order to eliminate secondary growths of organisms on the peptone the bacteriological examinations were made as soon as possible after the water was placed in the sample bottle.

The time interval between sampling and planting of samples never exceeded 10 minutes. Difco standardized prepared media was used, plates being incubated at 39°C. for 24 hours and 20°C. for 48 hours. Five portions of each dilution used were planted in lactose broth. Fermentation tubes showing any evidence of gas formation in 24 hours were streaked on E.M.B. plates. Those showing gas in 48 hours were also transferred to E.M.B. An organism was assumed to be B. coli if the E.M.B. transfer showed the characteristic appearance of the strains of organisms added to the raw water and no further confirmation tests were carried out. These growths on E.M.B. plates which differed in appearance from the stock strain (other strains naturally occurring in raw water) were confirmed in second lactose broth. The B. coli index was calculated by McCrady's method of most probable numbers.

Determinations of hydrogen-ion concentration, methyl orange alkalinity and turbidity were made on each of the samples of raw water used. Temperature observations were made at each stage of the work.

Determinations of residual chlorine were made by use of the standard method of A. P. H. A. employing standard ortho-tolidin solution. Readings on the samples containing both ammonia and chlorine were taken 20 minutes after the addition of the ortho-tolidin, and those with chlorine alone after 10 minutes. If the temperature of the solution was less than 15°C. the reading was taken at the maximum color development.

LAG IN BACTERICIDAL VELOCITY

The results of the studies show that within the limits of the experiments and for the same chlorine dosage there is a marked retardation in bactericidal velocity when the water is treated with ammonia and chlorine, as compared with water treated with chlorine alone, the lag being more pronounced with increased application of ammonia. However, after the initial retardation, the ultimate efficiency is greater in ammoniated water than otherwise and increases to some extent with increasing dosages of ammonia.

In experiment series "A" the chlorine applied was varied between 0.05 and 0.4 p.p.m., and the ammonia from 0 to 0.3 p.p.m. The work was performed at room temperature. The effect of the various combinations of ammonia and chlorine on the bactericidal velocity and efficiency of the treatment is shown in table 2. Each of the

TABLE 2

Data on bactericidal efficiency—experiment series "A"

(Each of the values is the average of the results of 5 tests)

NH$_3$ APPLIED, P.P.M.	CONTACT PERIOD	CHLORINE APPLIED, P.P.M. 0.05 — % B. coli remaining	Res. Cl$_2$, p.p.m.	0.10 — % B. coli remaining	Res. Cl$_2$, p.p.m.	0.20 — % B. coli remaining	Res. Cl$_2$, p.p.m.	0.30 — % B. coli remaining	Res. Cl$_2$, p.p.m.	0.40 — % B. coli remaining	Res. Cl$_2$, p.p.m.	Temp., °C.	Turb., p.p.m.	M.O. alk., p.p.m.	pHd
0	5 minutes	84.0	0.006	15.0	0.032	0.33	0.098	0.40	0.16	0.052	0.27	15.6	6.1	120	7.8
	30 minutes	79.9	0.003	2.7	0.018	0.047	0.062	0.56	0.11	0.0028	0.19	16.5			
	2 hours	75.6	0.002	3.9	0.010	0.016	0.038	0.0081	0.048	0.014	0.094	17.9			
	24 hours	52.3	0	0.54	0.001	0.24	0.006	0.20	0.011	0.014	0.040	19.2			
0.1	5 minutes	82.9	0.024	71.2	0.052	43.3	0.16	11.3	0.26	4.7	0.37	15.5	7.2	120	7.8
	30 minutes	79.1	0.005	75.2	0.029	8.9	0.13	0.92	0.23	0.26	0.34	15.9			
	2 hours	67.3	0.004	27.3	0.024	0.52	0.11	0.0039	0.20	0.0003	0.32	16.8			
	24 hours	17.0	0.001	10.4	0.010	0.00006	0.058	0.00002	0.13	0.010	0.25	19.4			
0.2	5 minutes	60.7	0.019	66.1	0.056	64.5	0.15	16.4	0.25	19.3	0.35	17.8	10.0	120	7.9
	30 minutes	82.7	0.012	41.5	0.042	30.5	0.13	12.2	0.23	0.99	0.34	18.4			
	2 hours	58.8	0.005	44.0	0.028	0.21	0.11	0.0035	0.22	0.0001	0.32	18.7			
	24 hours	2.9	0.001	0.0017	0.007	0.0002	0.066	0.000002	0.18	0.0013	0.28	20.2			
0.3	5 minutes	71.0	0.024	72.4	0.070	100.0	0.18	81.6	0.27	20.3	0.36	18.9	5.5	118	8.0
	30 minutes	87.8	0.016	67.0	0.054	27.2	0.14	14.3	0.25	6.8	0.35	19.6			
	2 hours	42.5	0.012	19.7	0.041	1.9	0.11	0.085	0.23	0.0081	0.34	22.6			
	24 hours	1.08	0.003	0.0004	0.031	0.0008	0.071	0	0.20	0	0.31	23.4			

values in the table represents the average of five separate tests.   The
data from table 2 are plotted in chart form in figure 1.   With 0.2
p.p.m. chlorine application the bactericidal effect obtained after 5
minutes, using no ammonia, is not equalled by the ammonia-chlorine
combinations until after 5 hours of contact.   A longer period of con-
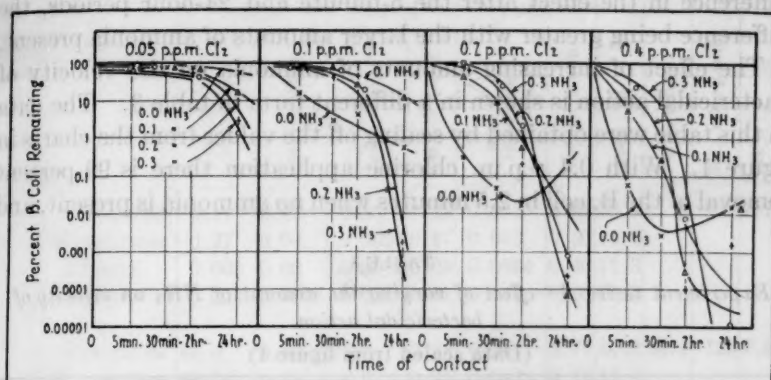
FIG. 1. EFFECT OF VARIATION OF TIME OF CONTACT ON BACTERICIDAL
EFFICIENCY OF AMMONIA-CHLORINE TREATMENT
Chlorine applied constant
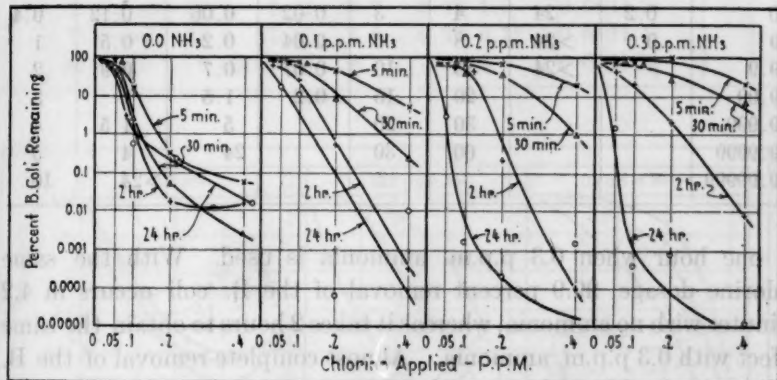
FIG. 2. EFFECT OF VARIATION OF CHLORINE APPLIED ON BACTERICIDAL EFFI-
CIENCY OF AMMONIA-CHLORINE TREATMENT
Ammonia applied constant

tact, however, results in greater bactericidal efficiency when ammonia
is used.   With 0.4 p.p.m. chlorine application the same general effect
is observed but the period in which the retardation in bactericidal
velocity occurs, due to the presence of ammonia, is shortened to 2

hours. This indicates that increasing the ratio of chlorine to ammonia hastens the bactericidal effect. Figure 2 has the same data plotted in a different form. With chlorine alone applied there is very little difference in the bactericidal efficiency between the 5-minute and 24-hour contact. However, with ammonia present there is a decided difference in the effect after the 5-minute and 24-hour periods, the difference being greater with the larger amounts of ammonia present.

The effect of increasing amounts of ammonia on the velocity of bactericidal action is shown in a different form in table 3. The data in this table were obtained by scaling off the values from the charts in figure 1. With 0.4 p.p.m. chlorine application there is 99 percent removal of the B. coli in 2.5 minutes when no ammonia is present, and

TABLE 3

Experiment series A—effect of varying the amount of NH_3 on velocity of bactericidal action

(Data scaled from figure 1)

PER CENT REMOVAL OF B. COLI	TIME REQUIRED TO OBTAIN BACTERICIDAL EFFECT, HOURS							
	0.1 p.p.m. Cl_2				0.4 p.p.m. Cl_2			
	0 NH_3	0.1 NH_3	0.2 NH_3	0.3 NH_3	0 NH_3	0.1 NH_3	0.2 NH_3	0.3 NH_3
90	0.2	24	4	3	0.02	0.06	0.12	0.4
99	9	>24	8	6	0.04	0.2	0.5	1
99.9		>24	15	10	0.07	0.7	0.9	2
99.99			20	15	0.2	1.5	1	3
99.999			30	20		5	1.5	4
99.9999			60	30		24	4	5
99.99999							>24	10

in one hour when 0.3 p.p.m. ammonia is used. With the same chlorine dosage, 99.9 percent removal of the B. coli occurs in 4.2 minutes with no ammonia, whereas it takes 2 hours to obtain the same effect with 0.3 p.p.m. ammonia. Almost complete removal of the B. coli is obtained in 10 hours with 0.3 p.p.m. ammonia and 0.4 p.p.m. chlorine, but it takes over 24 hours to obtain the same effect with the same chlorine dosage and only 0.2 p.p.m. ammonia.

A similar retardation in bactericidal velocity is shown by the results in experiments series "B." In this series the chlorine applied was varied from 0.2 to 0.8 p.p.m. and the ammonia from 0 to 0.4 p.p.m. The temperature range was from 1.5° to 23°C. The average results are summarized in table 4 and are plotted in figure 3.

TABLE 4

Experiment series "B"—data on bactericidal efficiency

(Each of the values is the average of the results of 6 tests)

| NH₃ APPLIED, P.P.M.	CONTACT PERIOD	CHLORINE APPLIED, P.P.M.						AVERAGE PHYSICAL AND CHEMICAL PROPERTIES			
		0.2 Cl₂		0.4 Cl₂		0.8 Cl₂					
		% B. coli remaining	Res. Cl₂, p.p.m.	% B. coli remaining	Res. Cl₂, p.p.m.	% B. coli remaining	Res. Cl₂, p.p.m.	Temp., °C.	Turb., p.p.m.	M.O. alk., p.p.m.	pH
0	5 minutes	0.11	0.07	0.36	0.22	0.051	0.68	10.9	11.6	123	7.7
	30 minutes	0.27	0.04	0.0056	0.17	0.013	0.50	11.1			
	2 hours	0.005	0.03	0.0011	0.09	0.0064	0.40	11.3			
	24 hours	0.033	0.0033	0.022	0.01	0.014	0.05	11.3			
0.2	5 minutes	46.0	0.15	31.0	0.35	0.03	0.77	10.9	10.4	123	7.8
	30 minutes	36.0	0.15	1.6	0.35	0.0022	0.76	11.1			
	2 hours	3.4	0.14	0.017	0.34	0.0004	0.74	11.5			
	24 hours	0.0001	0.08	0.0001	0.31	0.00002	0.65	11.8			
0.4	5 minutes	63.8	0.12	57.2	0.33	16.3	0.69	11.2	10.8	122	7.8
	30 minutes	62.3	0.12	13.7	0.32	1.0	0.68	11.2			
	2 hours	18.8	0.11	6.0	0.32	0.11	0.65	11.5			
	24 hours	0.0013	0.09	0	0.29	0	0.60	11.3			
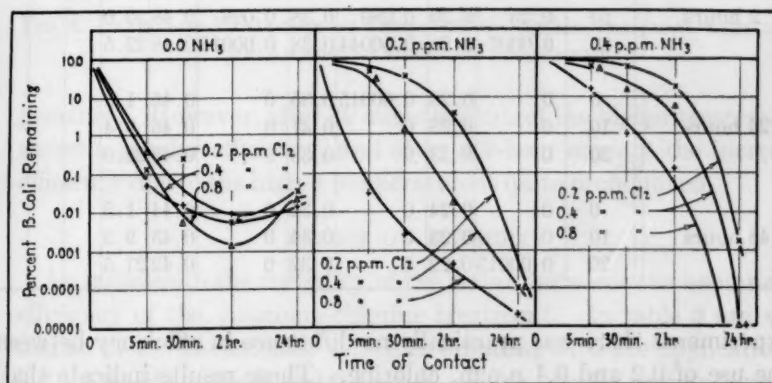
FIG. 3. EFFECT ON VARIATION OF TIME OF CONTACT ON BACTERICIDAL
EFFICIENCY OF AMMONIA-CHLORINE TREATMENT
NH₃ applied constant

When no ammonia is present we find no appreciable difference between the bactericidal efficiency of 0.2 p.p.m., 0.4 p.p.m. and 0.8 p.p.m. chlorine applied. The residual chlorine after 5 minutes contact with the 0.2 p.p.m. chlorine dosage was 0.07 p.p.m. and with the 0.8 p.p.m. chlorine dosage, 0.68 p p.m. Similarly in series "A"

TABLE 5

Experiment series C—effect of temperature on bactericidal efficiency

(Each of the values is the average of the results of 2 tests)

| CONTACT PERIOD | TEMP., °C. | NH3—Cl2 APPLIED, P.P.M. | | | | | | AVERAGE PHYSICAL AND CHEMICAL PROPERTIES | | | |
| | | 0.1 NH3—3 Cl2 | | 0.2 NH3—0.4 Cl2 | | 0.3 NH3—0.5 Cl2 | | | | | |
		% B. coli remaining	Res. Cl2, p.p.m.	% B. coli remaining	Res. Cl2, p.p.m.	% B. coli remaining	Res. Cl2, p.p.m.	Actual temp., °C.	Turb., p.p.m.	M.O. alk., p.p.m.	pH
5 minutes	0	21.9	0.29	35.9	0.39	56.7	0.49	1.3	5.6	123	7.8
	10	29.8	0.29	23.8	0.39	53.8	0.49	10.0			
	20	21.5	0.29	32.1	0.38	32.1	0.49	22.3			
30 minutes	0	6.1	0.29	52.1	0.39	8.6	0.48	1.5			
	10	3.8	0.29	50.7	0.39	5.2	0.48	10.0			
	20	0.90	0.29	0.20	0.39	0.28	0.49	23.5			
2 hours	0	0.22	0.28	50.1	0.38	2.9	0.48	1.3			
	10	0.25	0.29	0.089	0.38	0.096	0.48	10.0			
	20	0.0007	0.29	0.00044	0.38	0.00015	0.48	22.5			
24 hours	0	0	0.26	0.00015	0.35	0	0.46	1.2			
	10	0	0.25	0	0.37	0	0.46	9.4			
	20	0	0.23	0	0.33	0	0.45	22.0			
48 hours	0	0	0.24	0	0.34	0	0.44	1.3			
	10	0	0.23	0	0.34	0	0.43	9.3			
	20	0.00015	0.22	0	0.32	0	0.42	21.5			

experiments there was practically no difference in efficiency between the use of 0.2 and 0.4 p.p.m. chlorine. These results indicate that, within the limits of the experiments, the bactericidal effect obtained with a residual chlorine of 0.1 p.p.m. after 5 minutes contact is not improved materially when the residual chlorine is increased up to 0.7 p.p.m.

The bactericidal effect shown by the 37°C. and 20°C. bacterial counts was similar to that shown by the B. coli results. Therefore, in this paper only the B. coli counts will be considered.

EFFECT OF TEMPERATURE

The lowering of temperature of the water has a decided effect in reducing the bactericidal efficiency of the ammonia-chlorine treatment, as is shown by the results of experiment series "C" summarized in table 5 and plotted in figure 4. The temperature range was controlled between the limits 0° to 20°C., which represents the normal temperature variation in the Chicago water supply. After a 5-minute contact there is very little difference in bactericidal effect, due to tem-
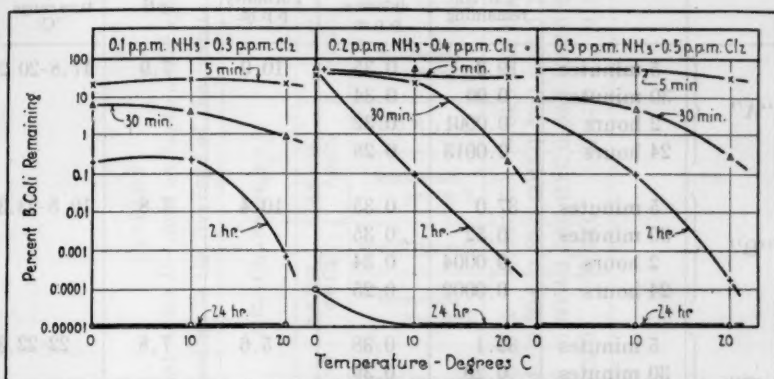
FIG. 4. EFFECT OF VARIATION OF TEMPERATURE ON BACTERICIDAL EFFICIENCY OF AMMONIA-CHLORINE TREATMENT

perature. However, after 30 minutes contact the higher temperature shows a greater efficiency, and after a 2-hour contact the increased efficiency due to the higher temperature is quite pronounced.

EFFECT OF TURBIDITY

An increase in the turbidity of the water decreases the bactericidal efficiency of the ammonia-chlorine treatment. In table 6 are summaries of the bactericidal velocities obtained with the application of 0.2 p.p.m. of ammonia and 0.4 p.p.m. chlorine under similar temperature conditions. In experimental series "A," "B" and "C," in which the average turbidity varied from 5.6 p.p.m. to 10.4 p.p.m., the percentage of B. coli remaining after 30 minute contact varied from

0.20 to 0.99 per cent. In experiment series "D" with the water having an average turbidity of 55.0 p.p.m. there was 15.4 percent of B. coli remaining after a 30 minute contact. A similar decrease in efficiency with increased turbidity was observed in a study recently made by the writer of data obtained during five years of chlorination at Chicago.[4]

TABLE 6

Study of effect of turbidity in water on bactericidal efficiency of ammonia-chlorine treatment

(0.2 p.p.m. NH₃—0.4 p.p.m. Cl₂ applied)

EXPERIMENT SERIES	TIME OF CONTACT	0.2 P.P.M. NH₃—0.4 P.P.M. Cl₂ APPLIED		AVERAGE PHYSICAL AND CHEMICAL PROPERTIES		
		% B. coli remaining	Residual chlorine, p.p.m.	Turbidity, p.p.m.	pH	Temperature range, °C.
"A"	5 minutes	19.3	0.35	10.0	7.9	17.8–20.2
	30 minutes	0.99	0.34			
	2 hours	0.0001	0.32			
	24 hours	0.0013	0.28			
"B"	5 minutes	37.0	0.35	10.4	7.8	19.5–23.3
	30 minutes	0.52	0.35			
	2 hours	0.0004	0.34			
	24 hours	0.0002	0.25			
"C"	5 minutes	32.1	0.38	5.6	7.8	22–22.3
	30 minutes	0.20	0.39			
	2 hours	0.0004	0.38			
	24 hours	0	0.33			
"D"	5 minutes	39.0	0.16	55.0	7.8	21.6–23.0
	30 minutes	15.4	0.14			
	2 hours	0.033	0.13			
	24 hours	0.0005	0.058			

EFFECT OF INCREASING CHLORINE DOSAGE

As is to be expected, increasing the ratio of chlorine to ammonia increases the bactericidal velocity, as is indicated in experiment series "D," summarized in table 7 and in figure 5. In this experimental series the ammonia applied was kept constant at 0.2 p.p.m. and the chorine varied from 0.2 p.p.m. to 2.0 p.p.m. A control with 0.4

[4] This JOURNAL, 23: 1, January, 1931.

TABLE 7

Experiment series D—data on bactericidal efficiency
(Each of the values is the average of the results of 4 tests)

NH₃—Cl₂ APPLIED, P.P.M.

CONTACT PERIOD	0.2 NH₃—0.2 Cl₂		0.2 NH₃—0.4 Cl₂		0 NH₃—0.4 Cl₂		0.2 NH₃—0.6 Cl₂		0.2 NH₃—0.8 Cl₂		0.2 NH₃—1.0 Cl₂		0.2 NH₃—1.5 Cl₂		0.2 NH₃—2.0 Cl₂		AVERAGE PHYSICAL AND CHEMICAL PROPERTIES			
	% B. coli remaining	Res. Cl₂, p.p.m.	% B. coli remaining	Res. Cl₂, p.p.m.	% B. coli remaining	Res. Cl₂, p.p.m.	% B. coli remaining	Res. Cl₂, p.p.m.	% B. coli remaining	Res. Cl₂, p.p.m.	% B. coli remaining	Res. Cl₂, p.p.m.	% B. coli remaining	Res. Cl₂, p.p.m.	% B. coli remaining	Res. Cl₂, p.p.m.	Temp., °C.	Turb., p.p.m.	M.O. alk., p.p.m.	pH
5 minutes	49.9	0.03	39.0	0.16	0.13	0.11	7.1	0.44	0.59	0.64	0.042	0.87	0.011	1.2	0.0027	1.6	21.6	55	122	7.8
30 minutes	55.8	0.02	15.4	0.14	0.035	0.04	0.094	0.38	0.0045	0.58	0.0065	0.77	0.00015	1.0	0.00015	0.79	22.0			
2 hours	20.8	0.012	0.033	0.13	0.068	0.03	0.0054	0.36	0.0002	0.52	0.0002	0.68	0.0001	0.84	0.000015	0.35	22.0			
24 hours	0.42	0.004	0.00005	0.058	0.0021	0.00020	0.00020	0.24	0	0.41	0	0.50	0	0.53	0	0.19	23.0			

p.p.m. of chlorine and 0 ammonia was included. The waters used were more turbid than those used in other experiments, the average turbidity being 55 p.p.m., the maximum 96 p.p.m. and the minimum 26 p.p.m. The work was carried out at room temperature. Each test was repeated four times. From figure 5 we find that the combination of 0.9 p.p.m. chlorine and 0.2 p.p.m. ammonia has no greater bactericidal efficiency in 5 minutes than has 0.4 p.p.m. chlorine alone. The combination of 0.3 p.p.m. chlorine and 0.2 p.p.m. ammonia has the same efficiency after 24 hours contact as has 0.4 p.p.m. chlorine alone. Unfortunately the combination of 0.9 p.p.m. chlorine and
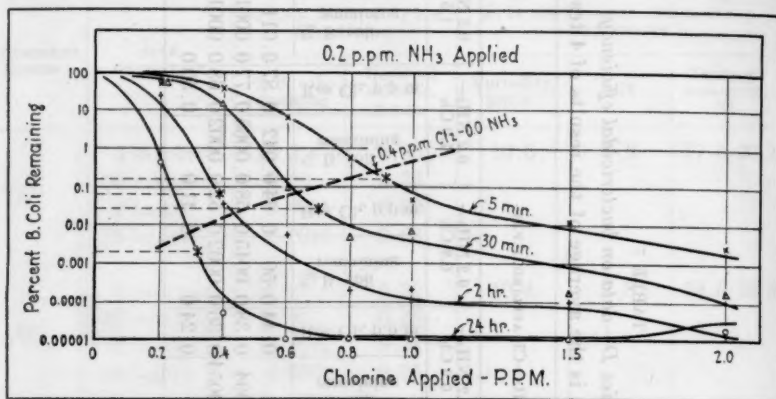
FIG. 5. EFFECT OF INCREASING OF CHLORINE APPLICATION ON BACTERICIDAL EFFICIENCY OF AMMONIA-CHLORINE TREATMENT

Ammonia applied constant

0.2 p.p.m. ammonia did not prevent the formation of chlorophenol tastes (table 9). We must, therefore, come to the conclusion that the bactericidal velocity obtained after a 5-minute contact with the chlorine alone cannot be equalled by the use of any combination of ammonia and chlorine without the formation of tastes.

PERSISTENCE OF RESIDUAL CHLORINE

The duration of the period in which residual chlorine (as shown by the ortho-tolidin test) persists in the water is greatly influenced by the amount of ammonia present and temperature of the water. Table 8 shows the effect of temperature and ammonia on the persistence of residual chlorine on a highly turbid water, 104 p.p.m. turbidity. At 20° and 22°C. the residual chlorine persisted approximately 2 hours

with 1.0 p.p.m. chlorine and no ammonia applied, whereas it persisted for 120 hours with the same chlorine dosage when 0.5 p.p.m. ammonia had been applied.   At 0° to 2°C. a trace of residual was still present after 24 hours when chlorine alone was applied, whereas with the addition of 0.5 p.p.m. ammonia and 1.0 p.p.m. chlorine there was 0.62 p.p.m. residual after 24 hours and as much as 0.2 p.p.m. residual after 22 days contact.

When the chlorine dosage is progressively increased in the presence of 0.2 p.p.m. of ammonia, a point is reached at which the residual chlorine breaks down and disappears more rapidly than if a lower

TABLE 8

Experiment series E—effect NH₃ and temperature on persistence of residual *Cl_2 in a turbid water**

(1.0 p.p.m. Cl_2 applied)

CONTACT PERIOD	RESIDUAL Cl_2 P.P.M. (O.T. TEST)					
	0–2°C.		10–12°C.		20–22°C.	
	0 NH₃	0.5 NH₃	0 NH₃	0.5 NH₃	0 NH₃	0.5 NH₃
5 minutes	0.60	0.65	0.60	0.60	0.60	0.65
30 minutes	0.40	0.65	0.45	0.60	0.40	0.65
2 hours	0.30	0.65	0.30	0.65	0.04	0.60
24 hours	Tr.	0.62	Tr.	0.55	0	0.45
48 hours		0.62		0.55		0.31
120 hours		0.55		0.45		0.03
264 hours		0.30		0.24		
480 hours		0.20		0.12		
528 hours		0.20		0.12		

* Turb. = 104 p.p.m.; pH = 7.6; m.o. alk. = 120 p.p.m.

dosage of chlorine were used or if the same dosage of chlorine and no ammonia were used.   This phenomenon may be observed in the results of experiment series "D" shown in table 7, and in experiment series "F" in table 9.   In each case the ammonia was kept constant at 0.2 p.p.m. and the chlorine applied varied from 0.2 to 2.0 p.p.m.

In experiment series "D" the residual chlorine after 24 hours contact, applying 0.2 p.p.m. ammonia and 1.0 p.p.m. chlorine, was 0.5 p.p.m.; while with the same amount of ammonia and double the chlorine application, 0.2 p.p.m. ammonia and 2.0 chlorine, the residual was only 0.19 p.p.m. after the same period of contact.   In experiment "F," 0.2 p p.m. ammonia and 2.0 p.p.m. chlorine gave a

TABLE 9

Experiment series F—effect of NH_3 in prevention of tastes produced by chlorination treatment

(Each of the values is the average of the results of 2 different tests)

NH₃ APPLIED, P.P.M.	PHENOL APPLIED, P.P.B.	CONTACT PERIOD	0.2 Cl₂ Taste	0.2 Cl₂ Res. Cl₂	0.4 Cl₂ Taste	0.4 Cl₂ Res. Cl₂	0.6 Cl₂ Taste	0.6 Cl₂ Res. Cl₂	0.8 Cl₂ Taste	0.8 Cl₂ Res. Cl₂	1.0 Cl₂ Taste	1.0 Cl₂ Res. Cl₂	1.5 Cl₂ Taste	1.5 Cl₂ Res. Cl₂	2.0 Cl₂ Taste	2.0 Cl₂ Res. Cl₂	Temp. °C.	Turb. p.p.m.	M.O. alk. p.p.m.	pH
0	0	5 minutes	0	0.08	1 Cl	0.25	1 Cl	0.50	1 Cl	0.63	1 Cl	0.80	2 Cl	1.0	2 Cl	1.5	20.5	11.8	120	7.7
		30 minutes	0	0.03	1 Cl	0.06	1 Cl	0.30	1 Cl	0.35	1 Cl	0.56	2 Cl	0.82	2 Cl	1.3	21.0			
		2 hours	0	0.01	0	0.04	0	0.10	0	0.25	1 Cl	0.45	1 Cl	0.71	2 Cl	0.98	22.7			
		24 hours	0		0	Tr.	0	0.01	0	0.03	0	0.05	1 Cl	0.31	1 Cl	0.60	23.0			
	20	5 minutes	1 CP	0.08	1 CP	0.28	1 CP	0.55	1 CP	0.65	2 CP	0.80	3 CP	0.90	4 CP	1.6	20.5	11.8	120	7.7
		30 minutes	1 CP	0.05	1 CP	0.08	1 CP	0.19	1 CP	0.38	2 CP	0.51	1 CP	0.78	2 CP	1.2	20.5			
		2 hours	1 CP	0.03	1 CP	0.07	1 CP	0.16	1–CP	0.20	1–CP	0.34	1–CP	0.68	1–CP	0.78	22.0			
		24 hours	1 CP	0	1–CP	0	1 CP	0.01	1–CP	0.03	0	0.05	0	0.25	0	0.49	23.0			
0.2	0	5 minutes	0	0.16	0	0.36	0	0.56	0	0.75	1–CL	0.96	1–CL	1.1	1–CL	1.6	19.0	14.0	122	7.8
		30 minutes	0	0.13	0	0.33	0	0.52	0	0.62	0	0.76	0	0.75	0	0.60	20.0			
		2 hours	0	0.09	0	0.30	0	0.48	0	0.56	0	0.69	0	0.60	0	0.23	20.0			
		24 hours	0	0.07	0	0.25	0	0.35	0	0.53	0	0.60	0	0.50	0	0.03	19.5			
	20	5 minutes	1–CP	0.15	1 CP	0.35	1 CP	0.55	1 CP	0.74	1 CP	0.93	2 CP	1.1	3 CP	1.5	19.0	10.4	122	7.8
		30 minutes	1–CP	0.12	1 CP	0.32	1 CP	0.51	1 CP	0.64	1 CP	0.75	2 CP	0.65	0	0.56	19.8			
		2 hours	0	0.11	0	0.32	1–CP	0.53	1–CP	0.62	1 CP	0.73	1 CP	0.63	0	0.31	20.0			
		24 hours	0	0.07	0	0.25	1–CP	0.45	1–CP	0.55	1–CP	0.64	0	0.57	0	0.08	19.8			

AVERAGE PHYSICAL AND CHEMICAL PROPERTIES

CHLORINE APPLIED, P.P.M.

	0.18	0	0.36	0	0.56	0	0.76	0	0.96	1 CL	1.4	1 CL	1.8	19.0	6.7	122	7.8
5 minutes	0.18	0	0.36	0	0.56	0	0.76	0	0.96	1 CL	1.4	1 CL	1.8	19.0	6.7	122	7.8
30 minutes	0.14	0	0.33	0	0.55	0	0.64	0	0.81	0	1.3	0	1.6	20.0			
2 hours	0.12	0	0.31	0	0.54	0	0.60	0	0.75	0	1.2	0	1.5	20.0			
24 hours	0.07	0	0.25	0	0.49	0	0.55	0	0.65	0	0.9	0	1.0	22.5			
5 minutes	1 CP	0.18	2 CP	0.36	2 CP	0.54	2 CP	0.74	3 CP	0.92	3 CP	1.3	3 CP	1.7	21.0	6.3	122	7.8
30 minutes	1 CP	0.13	1 CP	0.32	1 CP	0.52	2 CP	0.69	2 CP	0.82	2 CP	1.2	2 CP	1.6	22.0			
2 hours	1−CP	0.11		0.30	1 CP	0.50	1 CP	0.64	1 CP	0.79	1 CP	1.1	2 CP	1.6	21.8			
24 hours	1−CP	0.05	1−CP	0.23	1 CP	0.45	1 CP	0.60	1 CP	0.71	1 CP	0.91	1 CP	1.2	22.0			

Note. Intensity of taste recorded in Whipple's nomenclature. CL = chlorinous taste. CP = chlorophenol taste.

residual chlorine of 0.03 p.p.m. after 24 hours; while 2.0 p.p.m.
chlorine without ammonia gave a residual of 0.6 p.p.m.

Studies made indicate that the presence of the normal chemical
constituents in the raw water may be the cause of the break-down of
the residual chlorine under the previously mentioned conditions.
Parallel tests using distilled water, raw lake water and filtered lake
water show the same depression of residual when 0.2 p.p.m. ammonia
and 2.0 p.p.m. chlorine were applied to the raw water and filtered
lake water, but no such abnormal depression of residual when the
same amount of chlorine and ammonia was applied to the distilled
water.   There are indications that the break-down in residual occurs
when the ratio of chlorine and ammonia approaches 10 to 1.   With
0.5 p.p.m. ammonia present no such effects were observed when
3.0 p.p.m. chlorine was applied to raw lake water.

A number of tests were made to determine whether the form in
which the ammonia was applied had any influence on the persistence
of residual chlorine.   When equivalent amounts of NH_3 from various
ammonium compounds such as ammonium chloride, ammonium sul-
phate, ammonium carbonate, ammonium alum and ammonium
hydroxide, were used there was practically no difference in the amount
of residual chlorine remaining after a 24 hour contact.   For periods of
contact greater than this the results were very inconsistent and
require further study before definite conclusions can be arrived at.

TASTE AND ODOR PREVENTION

In general, our tests confirm the experience of others that the
ammonia-chlorine treatment will prevent taste and odors which are
formed or accentuated by the use of chlorine, but will not remove or
reduce the tastes which were already present in the water before
chlorination.

In experiment series "F" the effect of ammonia in the prevention of
chlorinous and chlorophenol tastes is studied by varying the amounts
of ammonia and chlorine applied to water containing approximately
20 parts per billion of phenol and to the same water without phenol.
The results of the tests, which were repeated twice, are summarized in
table 9.   Each taste result represents the average of the observations
of three technicians.

Chlorinous tastes which are produced by the application of 0.4
p.p.m. chlorine are prevented when 0.2 p.p.m. ammonia is present up
to a chlorine dosage of 2.0 p.p.m.   It took approximately a 1 to 1

ratio of ammonia to chlorine to prevent chlorophenol taste under the conditions of the experiment.

CONTROL BY RESIDUAL CHLORINE TEST

Caution should be employed in using the residual chlorine test (shown by ortho-tolidin method) for controlling the ammonia-chlorine treatment. The same residual chlorine test may be obtained by using different ratios of ammonia and chlorine, yet these various combinations of ammonia and chlorine may have widely varying bactericidal efficiencies. This is shown by the data in table 2. With 0.4 p.p.m. chlorine and 0.1 p.p.m. ammonia applied the residual chlorine test after 30 minutes contact was 0.34 p.p.m. and there was only 0.26 percent of the B. coli remaining. With the same chlorine dosage and 0.3 p.p.m. ammonia applied, the residual test after the same period of contact was practically the same, 0.35 p.p.m., but 6.8 percent of the B. coli remained. Without ammonia, 0.19 p.p.m. residual chlorine after 30 minutes contact left but 0.003 percent B. coli, whereas with 0.3 p.p.m. ammonia applied the same chlorine dosage showed 0.35 p.p.m. residual, but 6.8 percent of the coli remained alive. It is apparent that with the ammonia treatment higher residual chlorine values should be carried as a general rule.

CONCLUSIONS

The conclusions drawn from the data presented in this paper concern only the results obtained by the use of ammonia-chlorine treatment on Lake Michigan water having a hydrogen-ion concentration of 7.8 to 8.0. They are as follows:

1. The treatment is effective in preventing the formation of the chlorinous and chlorophenol tastes and odors which are associated with the chlorination of water supplies, and in addition is very efficient bactericidally where adequate contact is available.

2. For the same chlorine dosage the bactericidal velocity of ammonia-chlorine treatment is decidedly less during the first two hours after the treatment is applied than that obtained with chlorine alone. The larger the amount of ammonia present, the greater the lag in bactericidal effect.

3. For periods of contact longer than two hours the bactericidal effect of ammonia-chlorine treatment is greater than that obtained by the use of the same amount of chlorine alone. In this period the ultimate effect is greater with a larger amount of ammonia present.

4. By increasing the ratio of chlorine to ammonia it is possible to obtain a combination which will equal the bactericidal velocity attained after a five minute contact when using chlorine without ammonia. This ratio of chlorine to ammonia, however, did not prevent the formation of chlorophenol tastes and odors in the Chicago water.

5. The temperature of the water has a marked effect on the bactericidal efficiency of the treatment, the efficiency decreasing with lowering of temperature within the range of the experimental work, 20° to 0°C.

6. An increase in the turbidity of the water naturally decreases the bactericidal efficiency of the ammonia-chlorine process.

DISCUSSION

L. H. Enslow:[5] The paper by Mr. Gerstein covers a careful investigation of an important subject. The report from the Chicago laboratories discloses much of importance concerning the bactericidal efficacy and shortcomings of the rapidly advancing ammonia-chlorine treatment. I should like to congratulate Mr. Gerstein and his co-workers who made possible this paper and commend it to everyone interested in water purification for careful study.

There are two or three facts which stand out in the observations made in the Chicago studies.

a. The early conclusions of Sir Alexander Houston[6,7] showing that roughly 6 hours contact period may be required when using the ammonia-chlorine process to equal in efficiency the bacterial reductions secured in 30 minutes from chlorine alone, seem to be confirmed. I refer to the curves in figure 1 of Mr. Gerstein's paper in which dosages up to 0.2 p.p.m. chlorine were employed with from 0 to 0.3 p.p.m. ammonia expressed as (NH_3). With 0.2 p.p.m. chlorine alone results at 30 minutes were equal to those at 6 hours with ammoniation, regardless of the fact that the residual chlorine values were more than twice as great (0.13 p.p.m.) when ammoniation had been employed.

b. That the lag in bacterial reduction when employing ammoniation

[5] Research Engineer, The Chlorine Institute, Inc., New York, N. Y.

[6] Sir Alexander Houston: 19th, 20th and 21st Annual Reports to The Metropolitan Water Board, London, 1925, 1926, 1927.

[7] Linn H. Enslow: Jour. Amer. Water Wks. Assn., Dec., 1928, pp. 819–846. Munic. News and Water Wks., 75: 71, August, 1928.

can be reduced to a marked degree by increasing the chlorine dosage is proven and again confirms the early findings of Houston which have appeared from time to time in the London Metropolitan Water Board Reports.[6,7] The Chicago investigations indicate, however, that in no event does the contact period which would seem to be essential for safe practice drop below 2 hours. When the chlorine dosage was increased to double that required to produce without ammonia a satisfactory result within 30 minutes, then the contact period to produce about equal results with ammonia did drop to something less than 2 hours. All of this is well illustrated in the curves covering results from Series "A" and "B." In table 2 it is noted that 0.2 p.p.m. chlorine with 0.2 p.p.m. ammonia removes 99.79 percent of the B. coli group after 2 hours contact, but only 69.5 percent in 30 minutes. Increasing the chlorine by 100 percent (i.e., to 0.4 p.p.m.) gave after 30 minutes of contact a reduction of 99.00 as compared with the 69.5 percent reduction. There were no examinations to disclose the minimum period at which 99.79 percent removal resulted when employing 0.2 p.p.m. chlorine and 0.2 p.p.m. ammonia, but the curves would indicate an hour possibly, i.e., at the temperature involved, which was above 10°C. Reduction of the ammonia to 0.1 p.p.m. and increasing the chlorine from the 0.2 dosage to 0.4 p.p.m. gave a reduction of 99.79 percent after 30 minutes at temperatures above 15°C. There is no evidence, however, that this efficiency would be reached in 30 minutes at lower temperatures.

c. The Chicago results also confirm the conclusions of Holwerda[8,9] which were based upon extensive tests comparing the ammonia-chlorine process with ordinary chlorination in the governmental water laboratories of the Dutch East Indies: viz., that *the temperature effect is a very important factor when* employing chloramine or ammoniation with chlorination. Particularly is this true when the pH value of the treated water is above the neutral point of pH 7. Above pH 7.5 the temperature drop below 20°C. is particularly effective in rapidly reducing the efficiency of bacterial reductions during moderate contact periods. The Chicago results seem to indicate nothing less than a 2 hour contact period is satisfactory between the temperatures of 0 and 10°C. regardless of the ratio of chlorine to ammonia em-

[8] K. Holwerda: Mededeelingen von den Dienst Volkesgezondheid, 17: 251, 1928, (Dutch Indies).

[9] R. M. Chapin: a. Jour. Amer. Chem. Soc., 51: 2112, July, 1929. b. Jour. Amer. Chem. Soc., 53: 912, March, 1931.

ployed—the best results being secured generally with the lowest ammonia ratio, viz., one-third the chlorine dosage. Temperature would not seem to be of importance where long contact periods are available, say 4 hours or more, and provided sufficient residual chlorine is maintained.

THE SIGNIFICANCE OF RESIDUAL CHLORINE VALUES

It is quite apparent from the Chicago findings, as is likewise evident in those noted in the reports of Holwerda,[8] that the ortho-tolidin test for control of chlorine dosage continues of value in chlorine dosage control when employing ammoniation. This is true, but only with the provision that the former standards of residual chlorine requirement, after the 10 minute or longer period of contact, be increased materially above the 0.1 p.p.m. figure which in most instances has proven a dependable index of effectiveness from ordinary chlorination and limited contact periods. In other words, the evident weaker action of residual chlorine present in combination with ammonia requires that the concentration shall be increased to a new figure; in fact, one several times greater than that which proves effective when the chlorine is present in a more aggressive form, i.e., as chlorine, hypochlorous acid or even as hypochlorite.

Much is heard of the reduction in the chlorine consumption which has apparently been possible when employing ammoniation. It is becoming increasingly evident that the reduction in chlorine dosage to maintain a residual equal to that previously carried prior to introducing ammoniation constitutes a dangerous practice. I refer to previously published data by Holwerda[3] and now further confirmed by the Chicago studies. As examples of the point raised: Examination of the Chicago data shows residual chlorine values of 0.04 to 0.07 p.p.m. with ordinary chlorination employing 0.2 p.p.m. dosages are more effective than residuals of 0.68 p.p.m. (30 minute contact) when employing 0.4 p.p.m. NH$_3$ and 0.8 p.p.m. Cl, i.e., 4 times the chlorine dosage. At temperatures lower than 10°C. it appears that even with 0.38 to 0.48 p.p.m. residual chlorine the bacterial reduction (97.3 per cent) after 2 hours of contact is markedly inferior to the 99.78 percent secured with 0.28 p.p.m. residual. In the latter case the chlorine applied was 40 percent less and the ammonia reduced to one-third the chlorine value. With no ammoniation, not more than 0.15 p.p.m. residual was required under like circumstances to secure reduction of B. coli exceeding 99.95 percent. It is evident that the residual

chlorine necessary and the contact required will vary with pH value and temperature of the water and also the ammonia-chlorine ratio involved. It must, in any event, be higher when ammonia is employed and contact periods of less than 6 hours must be depended upon.

One interesting point which attracts attention is the correlation between the rate of disappearance of residual chlorine and the rate of bacterial destruction which appears in the Chicago data. Reduced temperature and increased ammonia decrease the rate of residual chlorine dissipation and coincidentally reduce the speed of bacterial destruction. The results strongly support the hypothesis that an actual combination takes place between the available chlorine and the bacterial protoplasm and thus causes death by an actual chemical combination of chlorine with the organism contents rather than by poisoning or oxidation, the latter once being the accepted hypothesis. Perhaps even a stronger proof that actual chemical change occurs when chlorinating bacteria is to be found in the observations by Butterfield, Purdy and Theriault,[10] of the U. S. Public Health Service laboratories, that the majority of the bacterial bodies are lysed and can not be later recognized as bacteria when suspensions of the live organisms were chlorinated and then examined under the microscope. In parallel tests, where the samples had been sterilized by pasteurization, the bodies of the dead bacteria remained and interfered with biological oxidation, but this was not true with chlorination or filtration of portions of the same aqueous suspensions.

THE EFFECT OF HYDROGEN ION CONCENTRATION

The Chicago studies were conducted on water samples having initial pH values above 7.5 and in all fairness to the capabilities of the ammonia-chlorine process it must be said that results at pH values below 7.0 should show the process in a much more favorable light. In other words, the contact periods required would be materially curtailed at values under pH 7.0. At pH values below 6.8 the rapidity of disinfection is as markedly increased as it is retarded at values of 7.5 and above. Holwerda[8] reports residual chlorine of 0.02 p.p.m. at pH 5.9 equally as effective as ten times that concentration at pH 9.0. Above pH 8.0 chloramine is extremely slow to disinfect and particularly so at low temperatures.

[10] C. T. Butterfield, W. C. Purdy, and E. J. Theriault, Public Health Reports, U. S. Treasury Dept., 46: 393, February 20, 1931.

A point of importance in this connection is the possible hazard of adding alkali to chlorinated water containing ammonia.   Chapin[9][11] has shown that the pH value of ammoniated-chlorinated water can be lowered with all of the advantages and none of the disadvantages which may attend the reverse process such as is practiced in raising the alkalinity in corrosion control at water works.   Chloramines may be broken down when alkali is added.   In consequence there will be a loss of available chlorine which may possibly be of some moment. One should watch for this in practice where the pH is increased following chlorination.   The chlorine residual should be checked beyond the lime application.   There is already existing evidence that lime application decreases the residual chlorine in the water beyond the point of "limitation."

In the ammonia-chlorine process the all important factors are the pH and temperature values.   Of less importance, perhaps, is the ratio of chlorine to ammonia applied.   With these factors varying the residual chlorine required and contact period necessary will also vary.   On this ground I feel that a note of warning should be broadcast concerning the necessity for recognizing that new and probably higher residual chlorine requirements than those found satisfactory in ordinary chlorination practice are rather to be expected for safety where ammoniation is employed or where "limation" of the finished water is practiced for corrosion prevention.   The residual chlorine requirements in many instances will have to be interpreted anew when employing the progressive schemes of chlorination.   Certainly residual chlorine values should be checked against bacterial analyses until it has been proven that the residual chlorine test remains the reliable "measuring stick" that it has proven to be in ordinary chlorination.   It will be prudent to determine what new or revised residual chlorine values must be specified under the altered circumstances.

[11] Linn, H. Enslow: Water Works and Sewerage, 78: 55, March, 1931.

COMBATING TASTES IN WEST VIRGINIA WATER SUPPLIES IN 1930

By E. S. Tisdale[1]

It was inevitable that tastes and odors of all varieties should appear in public water supplies drawn from the rivers and streams of the drought-stricken areas of twenty-one states during the epochal drought of 1930. Particularly were public water supplies in the Ohio River Basin affected.

West Virginia, which lies for the most part in the territory from the top of the Appalachian Mountains westward to the Ohio River, is drained by fairly large streams with good sized watersheds. In the short time available to discuss some aspects of the epidemics of tastes which appeared in all sections of West Virginia, only a few typical cases can be brought out. Three different areas which have widely varying characteristics will be discussed:

1. The canalized Monongahela River system, which has its outlet at Pittsburgh, Pa.
2. The canalized Ohio River public water supplies derived from the three hundred mile stretch of river between Midland, Pa., and Huntington, W. Va.
3. The canalized Kanawha River system between the newly developing industrial areas, centering at Charleston, W. Va., and the Ohio River at Pt. Pleasant.

Let us consider before we go into these three river systems a few general considerations. Accurate records from the weather bureau station, which is operated at the Clarksburg water filtration plant, showed that the greatest drought ever experienced in West Virginia began in the early Spring of 1930. The amount of rainfall received by the state for the calendar year 1930 was but 59 percent of the normal value. This lack of rainfall was a definite factor in raising to the nth degree taste troubles in public water supplies, and affording plenty of opportunity for using the weapons which are now in process of development for fighting and conquering abnoxious tastes.

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What were some of the tastes which appeared and what were the types of treatment attempted and how successful were they in actual practice?   On these matters I shall report briefly and summarize the opinions expressed by the chemists and superintendents of the water purification plants in West Virginia.   The weekly summary reports show tastes to have been encountered of the following characteristics:

1. Medicinal	8. Decomposition
2. Tarry	9. Creosote
3. Algae	10. Chlorine
4. Musty	11. Sweetish
5. Rivery	12. Fishy
6. Gaseous	13. Sewage
7. Flat, limey	14. Woody

The weapons used in attempting to eliminate these odors and tastes and their modifications as shown on the reports, were as follows:

1. Ammonia
2. Permanganate
3. Aeration devices
4. Activated carbon
5. Superchlorination and dechlorination
6. Acid mine wastes of natural origin
7. Copper sulphate

NORTHERN SECTION OF WEST VIRGINIA

Considering two small cities with modern filter plants on the upper part of the Monongahela watershed, we find Elkins had no tastes or odors from September, 1930, to January, 1931, because their source of supply, the Tygarts Valley River, absolutely dried up and the river bed for several weeks served as a dusty highway.   Spencer, a small town near Charleston, W. Va., was in the same predicament for four months and at the latter point $16,000 was spent by the West Penn Power Company to purchase six tank cars of water daily at Parkersburg and transport this water, by rail, 50 miles to Spencer where it was pumped into the city mains.   Tastes and odors disappeared at these two cities simultaneously with the drying up of the source of supply.

Weston, experiencing vile tastes and odors from algae and excessive sewage pollution, spent its energy not trying to eliminate tastes, but

in pumping out pools up stream for a distance of 9 miles.   The water was made safe by high chlorine dosage, the water company enduring the disagreeable tastes without an attempt to control them by any of the weapons just cited.   It is significant that travelers using the supply temporarily suffered intestinal disorders.

Clarksburg, 30 miles down stream with several impounding dams between Weston and the water intake, gave a history of excessive mineralization, but the disinfecting action of the mine wastes of Brown's Creek entering one of the raw water storage reservoirs, was exceedingly beneficial, in the opinion of the chemist in charge.   He states, "I really think that our water was a good example of the bettering effects of mine waste when flowing into a water whose river bed is largely limestone.   Indeed I felt last summer that Brown's Creek was our salvation, not only in increasing our flow, but in preventing our having the obnoxious tastes that so many had, some for the first time."   Mine wastes operated in the rôle of copper sulphate in nullifying algae growths in raw water storage basins.

Exceeding Clarksburg considerably in mineral hardness and acidity, came Fairmont on the Monongahela with a hardness of 634 and high mineral acidity on account of drainage of chemical wastes and acid mine discharges into the river pool.   Here the water in the mains was unpalatable with a flat limey taste.   No wonder!—at times as high as 2 tons of lime a day was used to treat 2 million gallons of water.   At times this required manual cleaning out of heavy sludge deposits in the sedimentation basin weekly.

Morgantown reports definite action with known weapons for controlling tastes.   When in January the Monongahela, after many months of stagnation, began to move, the water company first reported a "tar" odor.   Nuchar was resorted to, mixing it with the lime and feeding it into the raw water.   The taste improvement was not satisfactory so that the reserve mountain gravity supply was swung into service bringing into the city mains water with a decided "algae" taste.   Finally, by a combination of Nuchar and ammonia the river water supply was made reasonably satisfactory using 0.31 g.p.g. of Nuchar and 0.5 g.p.g. of ammonia.   It was estimated that 70 percent of the taste was removed from the river water by the combined treatment.   During the past two months the river water took on a musty taste which the combined use of Nuchar and ammonia overcame successfully.   It was estimated by the superintendent in charge that better results would have been achieved by applying the

Nuchar in advance of the chemicals. Had the by-product coke plant above Fairmont been pouring phenol into the stream as in former years, before the Interstate Stream Conservation Agreement of 1924 became effective, it is doubtful whether the Morgantown water could have been made potable by any known taste control method.

OHIO RIVER SUPPLIES—WESTERN SECTION OF STATE

Let us turn for a few minutes to a consideration of the public water supplies in West Virginia, along the Ohio River from the Wheeling area to Huntington. During the long period from February, 1930, to December, 1930, very slight or no troubles were noted. The superintendents in charge of filter plants were cautioned by a general warning sent out from State Health Departments about Thanksgiving time that troubles might be looked for when the fall rains set in. Troubles and tastes began to be reported the latter part of November, 1930.

Wheeling did not begin to use ammonia until December, 1931, after experiencing tastes which ranged from "slight" medicinal to "very pronounced" medicinal, then shifting to "musty-rivery." For ten days after ammonia was started the treatment was ineffective, but then it conquered entirely medicinal tastes. The chemist reports, however, that "you will note, however, that this treatment was not successful in removing other tastes which occurred in the water."

The superintendent of filtration notes that a great amount of bottled water was sold during late November and December and the sales fell off the later part of January. Six weeks was the approximate duration of taste troubles.

Other cities on the Ohio to attempt taste control were Weirton, Sistersville, Moundsville and Huntington. Weirton reports the use of permanganate starting early in January and continuing at a dosage of 1.4 p.p.m. until January 9 when both treatment with permanganate and ammonia were used. Ammonia dosage was at a slightly lesser rate than with chlorine, that is, 1.7 p.p.m. chlorine and 1.2 p.p.m. ammonia. On January 22 the permanganate treatment was discontinued. Later on the ratio of ammonia to chlorine was changed and through February and March, the ratio has been 1 part ammonia to 2 parts chlorine, which ratio has been effective in keeping down taste troubles at Weirton.

Sistersville used ammonia to some advantage, but non-familiarity with proper application produced low efficiency in taste control.

Moundsville, deriving water from wells in the river, experienced vile tastes which alarmed the people on January 6, 1931. After a hurried all night trip from Charleston the state's emergency ammoniator was installed by the State sanitary engineers, dosing at a rate of 0.3 p.p.m. Good results were secured, tastes disappearing immediately.

Huntington, a private company, one of the "American" chain, discarded ammonia experimentation about January 30, 1931, after a few days trial. The chemist then started successful use of Nuchar in combating stagnant, woody, musty, tastes and odors. Nuchar was used at a rate of 0.35 p.p.m. all through February and March and discontinued on March 25 when river water condition had improved considerably. In the opinion of the writer the "phenol" contamination of the Ohio River was almost infinitesimal during the runoff of January and February, 1931, when the flushing out of the eight month accumulation of wastes took place in the Ohio River Basin compared to what would have been the condition had such a drought occurred in 1925, before the remedial work of the Interstate Stream Conservation Agreement for the elimination of by-product coke oven wastes had been done. Each one of the product coke plants on the Ohio River watershed, whether the plant is in West Virginia, Ohio, Pennsylvania, Kentucky or the other six states in the Agreement, has provided for efficient disposal methods of phenol, formerly discharged to the river, hence the lessening of the chloro-phenol tastes in this flushout of the pools with the marked trend toward a decomposition type of taste and odor which was a source of trouble everywhere.

The musty-woody taste was the prevailing one of 1930 while in 1924 the major complaint was of bitter medicinal chloro-phenol tastes of a nauseating character which were about to destroy the public water supplies of Huntington and Wheeling, W. Va., Cincinnati, Ohio, and Louisville, Ky., and other Ohio River supplies. The eleven states, party to the agreement for administering a standard policy of phenol wastes control, have been successful in removing probably more than 95 percent of the phenol wastes formerly discharged into the raw water storage reservoirs of the cities lying in Ohio River drainage basin. It appears to be practical and possible to keep the accidental discharges to a minimum by close observation and cooperation with the by-product industry and to reduce the taste complaint caused by accidental discharges of phenols by ammonia treatment at the water works purification plants.

Now that it has proven possible to reduce phenol pollution on the Ohio shed it remains for sewage and other industrial waste control to be brought about before another such epochal drought appears on the watershed.

SOUTHERN SECTION OF WEST VIRGINIA

In speaking briefly of the happening on the third canalized river system, the Kanawha, where most unusual taste conditions prevailed for many months, together with certain other manifestations of intestinal disorders, we shall further emphasize the need for inaugurating a successful approach to stream pollution control.

Charleston, W. Va., lies at the mouth of the Elk River. The water works intake is in the Elk River pool about $1\frac{1}{4}$ miles from the mouth of the river. Only on rare occasions in the past has the Elk River flow been reversed and then only for a short time with Elk River low and floods on the Kanawha.

However, with the gradual drying up of the Elk River's 1550 square miles of drainage area in 1930, with the increase in the algae content of the stream from the accumulation of sludge deposits flushed from the sedimentation basin around the intake and the increasing amount of sewage and garbage-laden lower Elk and Kanawha River waters moving up stream from the main river, most unusual conditions occurred, as follows:

1. The dissolved oxygen content of the water dropped to zero at times.
2. Algae grew to unheard of proportions.
3. Daphnia and the succeeding cycles of protozoa and crustacea succeeded the algae growth.
4. The chlorine demand of the water was as great as that of many sewages due to the putrefying organic matter contained in the water. As high as 20 p.p.m. of chlorine were used to obtain a residual of 0.3 p.p.m. in the final filtered water.
5. Nauseating, vile odors resulted in the water as it arrived at the city spigots, clear and free from B. coli and of low bacteria count, even though the river water had a count of 150,000 per cubic centimeter and B. coli present in 0.0001 cc.

Loud was the popular outcry and earnest were the joint efforts of water company, state and local health departments to cope with the ever present odor and taste which was intensified to nauseating porportions in the hot water systems.

Every known weapon of taste and odor control was tried: super-

chlorination and dechlorination with SO_2, ammoniation, copper sulphate treatment of river pool to reduce algae content, aeration and activated carbon treatment combined with the normal sedimentation and filtration equipment, but the load was too heavy and no treatment availed to completely remove the nauseating odor and taste.

We must conclude that there are concentrations of sewage and algae and vegetable contamination beyond which we cannot go. Hence, the need for the pollution of our streams to be reduced.

ODORS, TASTES, HIGH CHLORINE DEMAND AND INTESTINAL DISORDERS PHENOMENON

The intestinal disorders which were noted in Charleston, W. Va., simultaneous with the appearance of these odors and tastes at their worst cannot be ignored by an impartial investigator. The U. S. Public Health Service, at the request of the State Health Commissioner, W. T. Henshaw, sent Dr. M. V. Veldee and R. E. Tarbett to Charleston on Armistice Day, November 11, 1930, to investigate the situation and cooperate with the State Sanitary Engineering Division to take control measures to assure a safe public water supply. Since an unusual phenomenon took place at Charleston early in November, 1930, which was followed by similar occurrences on the Ohio River about six weeks to two months later, these facts will be briefly cited.

Early in November an acute illness appeared in Charleston reaching epidemic proportions quickly. In a population of 60,000 people, state and government epidemiologists estimated 9,000 people were affected. The intestinal disturbance was characterized by severe pain in stomach, nausea, vomiting, followed by violent diarrhoea. This very explosive and extensive outbreak of gastro-enteritis taking place in the city occurred at a time when the bacterial records showed a raw water pollution of over 150,000 bacteria per cubic centimeter and B. coli present in 0.0001 cc. So gross was the pollution around the water works intake that when the sludge beds on the bottom of the river were disturbed there was a violent ebullition of methane gas. It appears likely from the epidemiological study that some unknown substance in the nature of an intestinal irritant—not a bacterial infection—present in the public water supply was responsible for the outbreak in Charleston in November. The water supply intake was removed as quickly as possible to a zone of less pollution in the Kanawha River.

No appreciable flow took place out of Elk and Kanawha Rivers until late December or early January, 1931. Fish killings took place in the lower Kanawha when the water moved down stream. Complaints of the bad tastes in Huntington, W. Va., Ashland, Ky., Cincinnati, Ohio, and Louisville, Ky., public water supplies prevailed throughout the Christmas vacation period.

Furthermore, abnormal chlorine demands were noted at Pittsburgh, at Wheeling and down stream Ohio River water supplies, when waters stagnant behind the navigation dams for nine months began to move down stream. Probably the Charleston condition calling for a dosage of 20 pounds per million gallons to satisfy the chlorine demand was the greatest experienced among the cities mentioned.

During the first week of January, 1931, coincident with the bad tastes, cases of gastro-intestinal disturbance were noted at Huntington and particularly at Ashland, Ky., reaching a peak about January 10. The heavy general rains over this territory would have caused the outflow of the Big Sandy River just above Ashland during the first week of January.

Further investigation of public water supplies below the junction of the Big Sandy and Ohio by M. V. Veldee, U.S.P.H.S. epidemiologist, elicited the facts that Ironton, Ohio, Portsmouth, Ohio, and Cincinnati, Ohio, and Louisville, Ky., experienced intestinal troubles similar to those recorded at Charleston, W. Va., in November, 1930, but they appeared in the Ohio River cities about the middle of January, 1931 and were less severe than at Charleston.

Coincident with the intestinal disorders came reports of dead fish in the streams, Elk, Kanawha and Ohio Rivers, together with observations of nauseating odors and tastes of the "musty-woody" type, and records of unusual chlorine demand. For want of any better name many physicians diagnosed the intestinal disorders as "intestinal flu" both in Charleston, W. Va., and in Ohio and Kentucky cities along the Ohio below West Virginia.

Reference to the literature, article by W. H. Dittoe, Chief Engineer, Ohio Health Department, describing the Salem, Ohio, typhoid outbreak, indicates that the preliminary outbreak in September, 1920, before typhoid made its appearance in October, 1920, was characterized by doctors and local health officials as "intestinal influenza."

DANGERS OF EXCESSIVE LOADINGS ON FILTER PLANTS

To one who personally went through the drought difficulties at Charleston, W. Va., observed the coincidence with taste troubles and gross pollution and observed the occurrence of similar, but less violent and widespread outbreak in down stream Ohio River cities, it appears clear that there are limitations to sewage and industrial waste pollution beyond which we can not go and produce satisfactory, palatable public water supplies. Bacteriological controls are not a complete criterion for judging the quality of a public water supply. Odors and tastes may be significant guides to the presence of certain now unknown chemical compounds which can affect fish life detrimentally and can cause intestinal disturbances among users of the public water supply.

As a result of the 1930 drought experience in West Virginia certain lessons have been learned with respect to coping with tastes in public water supplies. The use of ammonia and Nuchar have been effective for handling certain conditions. The worth of the Interstate Stream Conservation Agreement on the Ohio River Basin has been demonstrated and the need of further speeding up the program of stream pollution control made more evident to water companies, industries and individuals who now realize what it means to go without water or to have an unpalatable or insufficient public water supply.

AMMONIA SALTS IN TASTE ELIMINATION

By Harry E. Jordan[1]

The use of the ammonia-chlorine process at Indianapolis was occasioned by two factors: first, the desire for the elimination of some rather subtle tastes in the finished water at various seasons of the year; second, the desire to retard or prevent during the later summer months bacterial aftergrowths in the sedimentation basin of the slow sand filter plant. Our experience in control of the latter problem is not sufficient to justify discussion here. We are quite well satisfied with regards to minimization of taste—the process has been effective.

It is important to point out that at no time prior to the use of ammonia has the supply been subject to continued objectionable tastes. In the extreme cold of midwinter, at times of high rainfall in midwinter or early spring, at times in early summer when the water temperature was rising, occasional consumer comments have been made. We believe that Indianapolis has a low rate of comment regarding taste, but not so low as to justify indifference to the conditions. So the ammonia process was studied.

The filtration system consists of two plants deriving their supply through a common intake structure. The supply leaves the intake through two 48-inch diameter pipes—one leading to each set of filters. The slow sand plant consists of 6 units of a total capacity of 36 m.g.d., the supply for which is first coagulated and then settled in a basin having a detention capacity of 1.25 days at maximum rates. During the recent months low demand has increased the detention up to 2.5 days. The rapid sand plant has a normal rating of 12 m.g.d. with six-hour settling basins. Recent reduction in demand has increased this detention up to ten hours. Coagulant is applied at the head of each 48-inch pipe and pre-chlorine about ten seconds flow down the line. All filtered water flows to a contact basin where the post chlorine is applied. Approximately thirty minutes elapse before the operator checks the residual chlorine and forty-five minutes before the water reaches the pumps.

[1] Sanitary Engineer, Indianapolis Water Company, Indianapolis, Ind.

Preammoniation was started on September 24, 1930, using anhydrous ammonia and a Wallace and Tiernan outfit. The mechanical performance of the unit was satisfactory and the process instantly showed its effects in changed operating conditions. A desire to reduce costs and eliminate the handling of an additional compressed gas led to consideration of ammonium sulfate. We were aware of the use of aqua ammonia in December, 1929, of ammonium chloride, May, 1930, and the sulfate in an emergency on April 30, 1930. Continuous use of the latter salt was not recorded in America prior to October 20, 1930.[2] On that date we changed to and continued to use it up to the present.

The desire to minimize cost was brought to a focus by our inability to obtain a price lower than 17 cents per pound for anhydrous ammonia during September, 1930, when our initial studies were being made. Our first open market purchases of small lots of ammonium sulfate were made at 12.8 cents per pound NH_3. Later purchases of larger quantities have been made on a 10.4 cents per pound NH_3 basis with information as to prices for larger quantities in more favorable locations at about 8 cents. Like many other heavy chemicals the quantity purchased at one time influences the cost.

Owing to the wide distribution of Ford Motor Company ammonium sulfate, purchases can be made in practically any locality whenever it is desired to study the process.   Our stock is purchased from the Citizens Gas Company of Indianapolis in 100 pound bags. Delivery of carload orders is made by truck to our plant by the vendor. The ammonia content averages 25.5 percent, moisture 0.5 percent, and free H_2SO_4, 0.04 percent. Phenol is found to the extent of 0.003 percent. It is important to note here, that in order to add from this source 2 parts per billion phenol (frequently taken as the taste producing minimum) at an 8 pound rate of application, the sulfate could contain 0.2 percent phenol. In other words the determined phenol density in delivered ammonium sulfate at our rate of application is one sixty-seventh of the minimum taste producing amount.   It is a question whether this amount of phenol would be objectionable in the presence of so much ammonia. The sulfate stores well and shows

[2] In the 19th (1924–5) Annual Report of the Metropolitan Water Board (London) reference is made to experimental use of ammonium sulfate. The 21st report makes reference on page 16 to plant scale use of the material. By 1928, it appears to have been the regular source of ammonia whenever the ammonia-chlorine process was required.

less tendency to cake than many other materials. The grain, uniformly somewhat larger than granulated sugar, would feed easily through dry feed equipment provided that reasonable care was used to break down the fragile crystal masses that sometimes are found. It suits our convenience to use a solution feed of the material. It dissolves rapidly and is mixed with the aluminum sulfate solution. The combination is fed into the raw water about ten seconds before the pre-chlorine dose is applied.

In terms of the total applied chlorine, both before settling and after filtration, the efficient chlorine-ammonia (Cl-NH₃) ratio is 2.25 to 1. This is a larger NH₃ dosage than has been used by some plants, but conforms to our experience during the period when anhydrous ammonia was being used. It is necessary that we make further study of the interval between application of the pre-chlorine and the ammonia before it can be assumed that the present ratio is the most economical and efficient.

The chlorination process is controlled by the plant operators on a titration basis. The limiting amounts of excess chlorine are set by the Chief Chemist. Chlorine-ammonia operating technique is not based upon pounds of chlorine or ammonia per day or per million gallons, but upon recorded residual chlorine at stated points in the process and the proportion of NH₃ required to conform to the above ratio in terms of total chlorine. Plant operators, who are not technically trained men, but mechanics who have passed by experience and promotion up to their present grade, carry on routine ortho-tolidin tests and by reference to standard tables, direct the adjustment of the rate of application of chlorine and ammonia.

Prior to the use of ammonia we were adjusting applied water chlorine to from 0.05 to 0.1 p.p.m. Effluent chlorine was adjusted to from 0.05 to 0.1 p.p.m. when the water temperature was above 60°F. and from 0.1 to 0.15 p.p.m. when the temperature was below 60°F. At the present the applied water chlorine remaining is adjusted between 0.1 and 0.2 p.p.m. and the effluent at 0.15. Each time an adjustment is made in the chlorine rate a corresponding change is made in ammonia dosage to maintain the predetermined ratio. All possible requirements are set up in tabular form so that the operator is required merely to correlate the total rate of flow with the sums of the pre- and post-chlorination rates to determine the ammonia requirement.

In a recent publication "Ammonia Chlorine Process for Water Treatment" the following statement is made:

"Quite a number of the recently surveyed water plants operate in this manner; first, ammonia is added to the raw water, followed by split chlorination, part of the chlorine being added immediately after the ammonia and part to the plant effluent. We believe that this is a questionable procedure as no assurance can be had that the excess ammonia after the first chlorine application will reach the second point of chlorine application. If it does not, the post chlorination is nothing other than straight chlorination and any phenols still present will be chlorinated by the second treatment. This is particularly frequent in those cases where the first treatment is not sufficient to carry a residual through the filters, thus restraining ammonia consuming organic life in the filter bed. Where it is not possible to preammoniate and prechlorinate sufficiently to carry an adequate ammonia residual through the filters and into the plant effluent postchlorination is necessary, and if checks to determine the ammonia content of the filtered water show frequent absence of ammonia the water should be post ammoniated prior to and in addition to being post chlorinated."

This statement appears worthy of rechecking. It is directly opposed to our experience and would appear to deserve analysis in the light of experience in other plants where the problems are more severe. At times when the slow sand filters have been carrying the entire load and the interval between prechlorination-ammoniation and post-chlorination was about two days, determinations of free ammonia in the filter effluent showed no such amounts as would be accounted for by the initial addition. But the effects of ammoniation, as manifested by increased chlorine tolerance in the finished water, were quite definite. In fact, the anhydrous ammonia when it was being used was applied only to the slow sand inflow, but the effects in increased post chlorine tolerance in the combined effluents were immediate when the flow-through time had elapsed.

This leads us to suggest that the effects of preammoniation are not limited by the presence of free ammonia and that while a free ammonia test may serve in part as a measure in a rapid sand plant, its absence cannot be taken dogmatically to indicate a need to re-ammoniate a filtered water before final chlorination. It is obvious that some other chemical determination is needed to demonstrate the presence of the nitrogen compound that is effectively modifying final chlorine doses.

Our attention has been called to possibility of infringement of U. S. Patent No. 15903. This relates to the preparation of chloramines by the "reaction between aqueous solutions of chlorine and an ammo-

nium salt." The application states that "If the mixture is allowed to stand for a period of a quarter of an hour to three quarters of an hour at ordinary room temperatures . . . . the reaction is complete." The statement also is made that "better results are obtained by using both the constituents in strong solution." Finally this statement is found: "The chloramine solution prepared in accordance with the present invention may be added to water to be treated by the usual methods of dosage such as are employed for the addition of other reagents in solution in water."

The above quotations clearly indicate that the patent is granted on a process that involves the production of chloramine from strong solutions with a considerable period allowed for completion of reactions. The completed material is supposed to be applied to the water under treatment rather than its components. In our use of the ammonium sulfate with chlorine, both are in dilute solution, and they are not combined outside the water to be treated. Clearly the intent of the applicant is in contrast to our practice. It is rather interesting to note that the patentee claims reacting value to ratios of 1 chlorine to from 0.07 up to 1 NH_4. That is a rather wide range and quite in contrast to the frequently expressed 4 to 1 or 3 to 1 ratios.

When the use of ammonium sulfate was first undertaken it was suggested to us that "it cannot be used effectively unless the water is soft, it cannot be used with water containing calcium, it is inclined to accelerate corrosion, the sulphuric acid evolved by hydrolysis causes rust to form quickly and proper diffusion may be difficult." Our supply is not normally termed soft—the calcium averages 72 p.p.m., but the results are definite and satisfactory. We note no corrosive tendencies—red water is not produced. The added SO_3 is less than 1 p.p.m. Diffusion is no more difficult than with any dissolved material.

The use of the sulfate unquestionably improved the taste of the water during the winter. During last autumn and winter, the low stream flow produced increased taste problems. Many cities in the middle west experienced a severe consumer reaction, with unfortunate and ill informed comment as to the safety of the supplies. With similar stream conditions, no such reaction occurred in Indianapolis. Chlorinous or musty tastes have not been present. Consumer comment has been reduced to a minimum. Bacteria removing efficiency of the plant has been increased for the persistence of titrable chlorine has had an increased destructive effect upon bacterial life. Organisms of

the colon group have been blanked out earlier in the purification process than was possible when chlorine alone was used.

As a matter of record, we propose to discontinue the use of ammonia shortly on the assumption that it is not necessary during the summer months.   We may find, however, that our patrons have been conditioned to it so that adverse comment will result upon its discontinuance.   If so, we will be forced to conclude that one more treatment material has been added to those which already contribute to the health and comfort of our customers.

AMMONIA-CHLORINE TREATMENT AT BEAVER FALLS AND NEW BRIGHTON, PA.

By E. C. Goehring[1]

The Beaver River, from which the Beaver Valley Water Company derives its supply of water, is a stream which fluctuates very widely in chemical characteristics. This is due mainly to the fact that the tributaries that make up this stream pass through densely populated sections, where the chief industries are those connected with the making of iron and steel products. The Beaver River has a drainage area of 3,040 miles, with three streams uniting to make up the entire flow.

Although the State Health Departments of the States of Pennsylvania and Ohio have made admirable progress in the elimination of the most objectionable wastes from the plants located in their territories bordering on these streams, there is still a chance for improvement in minimizing particular wastes, especially those of phenolic composition coming from by-product coke plants. These wastes although of gradually decreasing intensity during the past few years, still find their way into our raw water supply, and are those which cause the greatest difficulty in the operation of the filtration plants, because of the obnoxious medicinal flavor they impart to a chlorinated supply.

During the past five years, various treatments have been tried to eliminate this taste condition with little favorable success. In March, 1930, after a winter of unusually bad tasting water from these phenolic compounds, an ammoniator was installed. The Beaver Valley Water Company operates two plants within 3 miles of each other on the same stream, which makes an ideal condition for comparing the effluents of the two plants using different processes, as well as different modifications of the same process in respect to application points of the various chemicals used.

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PRELIMINARY EXPERIMENTS WITH CHLORINE AND AMMONIA

Although no phenols were present in the raw water when the machine was installed, it was nevertheless operated to ascertain its effectiveness in reducing other tastes and odors from algae, as well as its relative effectiveness of sterilization, as compared with the chlorine treatment alone. It was therefore planned to use the ammonia-chlorine treatment at the upper or Eastvale Plant, so that a comparison might be made of the effluent of this plant with that from the New Brighton Plant, operated without ammonia treatment. The ammoniator was installed to apply the ammonia to the discharge of the raw water pump, while at first the chlorine was applied to the filter effluent. At

TABLE 1

MONTH	EASTVALE PLANT			NEW BRIGHTON PLANT		
	Ammonia to raw water	Chlorine to filtered water	Ratio Cl₂: NH₃	Chlorine		
				Raw	Filtered	Total
March......................	0.236	0.322	1:0.733	0.128	0.259	0.387
April......................	0.214	0.362	1:0.592	0.143	0.306	0.449
May.......................	0.184	0.296	1:0.643	0.164	0.270	0.434
June.......................	0.143	0.264	1:0.542	0.173	0.230	0.403
Average...................	0.194	0.321	1:0.605	0.152	0.266	0.418

Results in parts per million.

the New Brighton Plant, at which no ammonia was applied, the sterilization was by pre-chlorination of the raw water, followed by post-chlorination of the filter effluent, which from past experience had been found the most effective treatment for the type of water then being treated.

The above scheme of treatment was in effect for the months of March, April, May and June, during which time the raw water was of as good a quality as is ever found in the Beaver River, the chlorine demand was at its lowest point, and no phenolic tastes or odors apparent. Table 1 shows the chlorine dosages at both plants, as well as the ammonia dosage and ratio used at the Eastvale Plant.

A comparison of the effluents of the two plants during these four months brought out the following conclusions:

(1) The ammonia treated water was by far more palatable than the pre- and post-chlorinated water.

(2) Samples of both effluents stored for long periods at both air and refrigerator temperatures showed the ammonia-treated water to remain more palatable and odorless than did the other water.

(3) Residual chlorine tests made in both systems showed this test appeared in all parts of the system where the ammonia-treated water was pumped, whereas the chlorine residual in the other system vanished within a short time after leaving the plant.

(4) This delayed effect in the disappearance of the chlorine residual with ammonia-treated water, brought out the fact that in the ammonia-treated system the reservoirs failed to show any recurrence of gas-formers or after growths, which were found on several occasions in the unammoniated water.

(5) Another observation made during this time which necessitated considerable effort and study was the fact that a decided lag was noticed in the sterilization rate in the ammonia-treated water. In order to make certain that sterile water left the plant, a series of inoculations were made directly after chlorination and at regular intervals thereafter, to determine how long before sterile water could be expected to leave the plant. After several tests it was found that with a contact time of two hours which our clearwell at this plant allows, it was necessary to carry a residual of 0.20 immediately after chlorination, which by the time it reached the tap had been reduced to 0.15 p.p.m. At the time these first tests were made, our pH value was about 7.2 on the tap water. Compared to this time necessary for complete sterilization with chlorine alone, it was found that but fifteen minutes were required, using the same residual chlorine content.

(6) It was also noticed that it was possible to carry a higher residual chlorine content with the ammonia-treated water without the formation of chlorinous tastes, whereas, at the plant using no ammonia, the residual chlorine content had to be controlled at a very definite point, or within a very much lower range, in order to prevent the appearance of chlorinous tastes and odors.

(7) From a comparison of the chlorine used at the two plants during the same period, the numerical values show more chlorine as having been used without the ammonia treatment than with it. In order that these figures might not be misleading, it must be recalled that in the case of the New Brighton Plant, both prechlorination and post-chlorination were in use, a procedure which generally requires a higher chlorine usage than post-chlorination alone, because of the necessity of satisfying the chlorine demand of many substances which are generally removed during sedimentation and filtration. It is, therefore, not a conclusive fact that ammoniation allows the use of less chlorine.

About the end of June, due to low and warm water conditions, algae began to form in the sedimentation basins and on the filters, more so in the ammonia-treated water prior to chlorination than in the

pre-chlorinated basins. We began to experience air-bound filter trouble, along with shortening of the filter runs and high wash water usage. It was, therefore, decided to investigate the effectiveness of the ammonia-chlorine treatment in overcoming this trouble. Subsequently, a chlorinator was installed to apply chlorine at the outlet of the mixing chamber to the pre-ammoniated raw water. At the same time, at the other plant, the chlorine dosage to the raw water was raised to a point where it was possible to carry a residual chlorine content as far as the filters, whereas before this residual was only traced as far as the first baffle. At both plants a residual chlorine was maintained to give a value of 0.20 p.p.m. on the filter beds.

This scheme of treatment was effected at both plants for the next four months, including July, August, September and October.

TABLE 2

| MONTH | EASTVALE | | | | | NEW BRIGHTON | | |
| | NH₃ | Chlorine | | | Ratio Cl₂: NH₃ | Chlorine | | |
		Pre-	Post-	Total		Pre-	Post-	Total
July	0.141	0.342	0.242	0.584	1:0.413	0.345	0.218	0.563
August	0.163	0.412	0.273	0.685	1:0.420	0.437	0.275	0.712
September	0.182	0.405	0.237	0.642	1:0.353	0.513	0.260	0.773
October	0.245	0.512	0.348	0.860	1:0.351	0.615	0.286	0.901
Average	0.183	0.418	0.275	0.693	1:0.379	0.477	0.257	0.737

Results in parts per million.

The chlorine dosages at both plants, as well as the ammonia dosage and ratio used at the Eastvale Plant; are shown in table 2.

Within a period of ten days after chlorination of the pre-ammoniated water was started, the algae growths on the sedimentation basin walls began to peel off in sheets, and the air-bound condition in the filter beds left, resulting in a reduction in filter washing and wash water usage. At the New Brighton Plant with pre-chlorination alone similar effectiveness in algae was noted, but at the expense of slightly more chlorine, as well as the appearance of a slight musty after-taste at the tap.

During the period represented in table 2, the raw water had considerable odors and taste, classed as earthy and musty. The drought conditions affected us to the extent that the Beaver River was at the

lowest flow that has been observed for some fifty years. The pollution load due to sewage and wastes other than phenolic, which are absent in our water during the summer, increased gradually, as can be seen from the increasing chlorine demand. Besides the conclusions noted with reference to the effectiveness in algae control, the other advantages already mentioned were also observed during this period.

OPERATIONS DURING PHENOL TROUBLES

Because of the effectiveness of the ammonia-chlorine treatment during the period of its use, and the approaching season of cold weather and water, during which it has been the normal procedure for trouble from phenols to appear, we decided to install an ammoniator also at the New Brighton Plant. This W and T Ammoniator was installed at this plant the later part of October to apply the ammonia to the discharge of the raw water pump. To duplicate the treatment scheme

TABLE 3

NOVEMBER	AMMONIA	CHLORINE			RATIO— CHLORINE TO AMMONIA
		Pre-	Post-	Total	
Eastvale........................	0.424	0.732	0.264	0.996	1:0.58
New Brighton...................	0.437	0.763	0.272	1.035	1:0.57

Results in parts per million.

in effect at Eastvale, a chlorinator was set up to discharge chlorine into the pre-ammoniated water at the mixing chamber.

During the month of November, both plants were operated as nearly alike as possible, both using pre-ammoniation followed by pre-chlorination, with post-chlorination at the filter effluent to give the desired residual for complete sterilization. The chlorine and ammonia used at both plants during this month, during which time both effluents were comparable as to palatability and sterility, are shown in table 3.

On December 7, without any notice, and at 3 o'clock in the morning, the first dose of phenol came. The chlorine demand, which had been gradually increasing all season, at the time was about 1 p.p.m. and within a few hours this demand rose to 32 pounds to the million gallons or nearly 4 p.p.m. Unfortunately, our ammonia machine was not capable of delivering sufficient ammonia to keep up the ratio we were attempting to keep, and the resulting ratio fell to 1:0.21::

chlorine to ammonia.   The resulting water at the tap was very offensive, and decidedly more noticeable to the average consumer, due to the improvement in palatability of the water that had been furnished the previous nine months.   Within a very short time we received another larger orifice, but not until the peak of the phenol spill had passed, which was evidenced by the decrease in the chlorine demand to about 1 p.p.m.

The ammonia dosage was increased to as high as 2.06 times the amount of chlorine being applied with no apparent relief.   Both plants were then being operated, using prechlorination of the ammoniated raw water, and the effluent from both plants was equally unsatisfactory.   To attempt to better the situation, it was decided to discontinue the prechlorination dose, and resort only to ammoniation of raw water followed by chlorination of the filtered effluent.   This was done at the Eastvale Plant, and within the time necessary for a complete change of water to pass through the plant, a decided improvement was noticed and within twenty-four hours the water was again quite satisfactory.   A similar change at the New Brighton Plant where the chlorine was applied to the water leaving the sedimentation basins, instead of to the raw water, brought about the same improvement.   Due to the construction of the New Brighton Plant clearwell, it was found that too short a contact period is available to insure sterile water leaving the plant, when the chlorine is added to the filter effluent, and by adding the chlorine to the filter applied water, it increased our contact time sufficiently to insure this.

The most important observation made during this period, during which phenols were present in the raw water, was that prechlorination of the ammoniated raw water is not effective, and the greater chlorine demand of the raw water produces unfavorable results in the taste and odor of the filtered water.   This observation was made during three different periods with the same results.   It was also observed that, with a water of considerable turbidity, such as was encountered during the week of April 2, 1931, more than three times as much chlorine and ammonia were necessary to maintain the same residual at the New Brighton Plant, at which we were running prechlorination to the ammoniated raw water; than was necessary at the Eastvale Plant during the same time, where we were applying the ammonia and chlorine to the effluent.   The taste at the Eastvale Plant was not noticeable, whereas the effluent of the New Brighton Plant was not satisfactory.   Here, again, as soon as the prechlorination dose was

applied to the settled water, after most of the turbidity and color had been removed by the settling process, a decided improvement as well as decrease in the chlorine and ammonia requirement was noticed.

After the first phenol trouble was over, we had a period extending from December 12 to January 7, during which the filtered water was quite satisfactory. The chlorine demand averaged about 1.25 p.p.m. and a ratio of chlorine to ammonia of 1 to 1 was found the most effective.

The second and worst dose of phenol came early on the morning of January 9, when the chlorine demand of 1.25 shot up to 6 p.p.m. The raw water had a most offensive odor of tar or phenol, which was aggravated by the fact that the river had been frozen over, and was at the time at very low flow. The ammonia, which was being applied to the raw water was applied to maintain ratios of chlorine to ammonia from 1 to 2.2, as high as 1 to 4, without any particular benefit. Samples taken for odor determination through the purification system showed a slight reduction in intensity in passing through the plant, but the filter effluent itself, prior to chlorination had a very distinct phenolic odor. The odor on the chlorinated water, in spite of the fact that as high as 6 p.p.m. of chlorine was necessary to maintain a residual, was no worse than the water prior to chlorination; which proved that the intensifying effect of chlorine had been removed. This experience was also verified in the previous phenol spill and in a later one, and showed that while ammonia treatment will not remove phenolic tastes and odors, it will effectively prevent them from getting worse. This is in itself a very commendable feature, in that it is common knowledge that chlorination intensifies phenolic taste as much as 50 times. Our experience has shown that with conditions as intense as we are subjected to, either of two things must be done to remove this trouble completely. The most obvious is to request a more thorough supervision and reduction of the phenols at the point of introduction to the stream, by the parties in charge; or to supplement our purification process with some form of treatment, such as activated carbon, to eliminate the phenolic compounds themselves.

The third and last dose of phenol came on February 16, three days after we had been notified by the Pennsylvania State Health Department that 5,000 gallons of phenols had been dumped into the Mahoning River at Youngstown, Ohio. Fortunately for us, the first effective rainfall of the season and the subsequent increase in river flow diluted this spill to the extent that it was hardly more than noticeable

in the raw water.   Compared to other chlorine demand peaks of 3.8 and 6.0 p.p.m., the peak during this spill was slightly less than 2 p.p.m.   The intensity of the odor was proportionate to the chlorine demand.   By maintaining a ratio of chlorine to ammonia of 1 to 2 during this spill, we were able to minimize the taste and odor on the tap water to the extent that it was hardly noticeable except at the very peak.   This experience proved to us that, if the phenolic compounds are not present in too large a concentration, and the resulting chlorine demand not too high, the ammonia-chlorine treatment will assist in minimizing the taste from chlorophenols.

CAUSES FOR HIGH AMMONIA USE

During the period of time covered by the foregoing data no time was available to study the reason for the obviously too high ammonia usage.   Because of the heavy pollution load present during the low water periods, priority was given to the study and maintenance of the bacterial purity of the water, and this phase more than took up the time available for laboratory investigations.   However, with the return of a more normal condition, a concentrated effort has been made to study the reasons for the high ammonia dosages necessary. Although the work on this problem is far from complete, observations made so far have brought out many points which, to my knowledge, have not appeared in any literature published on the ammonia-chlorine treatment.

Ammonia determinations, using direct Nesslerization, present the following conclusions:

(1) When the ammonia was applied to the raw water, reductions varying from 5 to 25 per cent were found in passing through the sedimentation basin.

(2) Reductions in free ammonia content of from 40 to 60 per cent were found to take place in the water passing through the filters. The reduction depends upon and varies with the rate of flow through the beds, as well as with the length of time the filter had been in service.   From the information on the loss through filters, it is obvious that the ammonia is used up by the bacteria, and as was noted at the Eastvale Plant, algae growths were more in evidence.

(3) Overall reductions in ammonia content from 40 to 70 percent were found in passing through the entire purification system, from the raw water to the filter effluent, when the ammonia was applied to the raw water and the chlorine not applied until the filter effluent.

The remedy for this loss or waste of ammonia is quite evident, and when the chlorine was applied shortly after the ammonia, or prior to the time the ammoniated water passed through the filters, a decided reduction in this loss was found.

(1) When the chlorine was added to the raw water shortly after the ammonia, reductions in free ammonia content through the basin dropped to between 0 and 10 percent.

(2) The reduction in free ammonia through the filters, when chlorine was applied to the ammoniated raw water, has been found to be less than 25 percent, and in some cases no appreciable loss could be detected.

(3) Overall reductions in ammonia content in passing through the entire settling and filtering process have varied with the amounts used, from 5 to 25 percent.

From these recent determinations, a decided economy in ammonia consumption, by the use of the proper application points of ammonia and chlorine, could have been obtained. This has been proved in recent operation, where the ammonia and chlorine have been applied close together, but far enough apart to insure complete mixing with the ammonia prior to chlorination.

CONCLUSIONS

In concluding this paper, I should like to mention briefly the most important observation made during the past year:

(1) The ammonia-chlorine process has proven itself valuable in minimizing chlor-phenolic tastes and odors. It will not remove phenols, but prevents their intensification upon chlorination.

(2) The ammonia-chlorine treatment when used in the raw water is an effective means of combating algae conditions in the basins and on the filters.

(3) With sufficient contact time, the ammonia-chlorine treatment is a more efficient sterilizing agent than chlorine alone.

(4) Higher residual chlorine content may be maintained with ammonia present, so that aftergrowths may be prevented by maintaining residual chlorine in the reservoirs and the distribution system, without the danger of producing chlorinous tastes and odors.

(5) When phenols are absent, and the raw water turbidity is low, or, in general, when the chlorine demand of the raw water is low, the best method is to apply the ammonia and chlorine to the raw water. The advantages of this system are: the effective control of algae in the

basins and filters; added safety in insuring sufficient contact time for complete sterilization, (this is especially desirable at plants where the pH value of the treated water is above 8.4, where iron and manganese removal necessitates such high pH values). Upon proper conditioning of the filter beds, only one dose of chlorine need be applied, as ammonia assists in carrying residual chlorine through the beds themselves without any appreciable loss.

(6) When phenols are present, or with turbid water, or with water which has a high chlorine demand, the more effective point of application of the ammonia and chlorine is at the filter effluent. With this scheme of operation, sufficient contact time in the clearwell must be allowed for a sterile tap effluent. High pH values must be guarded against, but they may be offset to some extent by the possible increase of residual chlorine content.

I wish to extend thanks and appreciation to Dr. Scmelke, and Dr. Baker, for their valuable suggestions and opinions, as well as for the service they rendered me in connection with the operation of the ammonia-chlorine treatment at our two Beaver Valley Water Company plants.

THE AMMONIA-CHLORINE TREATMENT AT CLEVELAND

BY W. C. LAWRENCE[1]

After conducting two years of experimental work on practically all kinds of activated carbons for the prevention of tastes in the Cleveland water supply, it was suggested by the laboratory that ammonia might be tried as a possible solution to the problem. This was thought advisable, especially since previous investigators had proven that ammonia in conjunction with chlorine would eliminate tastes and still be as effective as a sterilizing agent at a very nominal cost. After some two months research with ammonia, sufficient data were obtained to draw the attention of the Committee appointed by the City Manager to study and consider the activated carbon process, which led them to recommend this treatment as most feasible for the local conditions.

In starting the laboratory research work on this method of treatment, the studies conducted covered chiefly the value and limitations of the process as an effective remedy for taste prevention on Lake Erie water, and the possible use of ammonia in conjunction with chlorine in perfecting sterilization, along with a more thorough understanding of how these effects are brought about.

This preliminary work established certain effective ranges of concentration for phenol, ammonia, and chlorine; other factors such as temperatures and detention periods required in both taste prevention and sterilization were also studied. These results simplified the experimental plant operation. Although during all of the work with the experimental plant, laboratory work was carried on, either as an indication or control for the plan of procedure or to confirm results obtained on the larger scale.

The concentrations of phenol, or ammonia still waste liquor used, ranged from 0.1 to 1.0 p.p.m., the ammonia from 0.25 to 0.5 p.p.m., and chlorine from 0.3 to 2.5 p.p.m. Detention periods ranged from 0.5 to 24 hours. The water used in all this work was filtered water unchlorinated.

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The results obtained in conducting this laboratory work were in general as follows: (1) Tastes were prevented with 0.25 p.p.m. of ammonia and ranges of chlorine from 0.2 to 0.5 p.p.m., when the concentrations of phenol ranged from 0.1 to 0.5 p.p.m., and when 0.5 p.p.m. of ammonia was used with ranges of chlorine from 0.3 to 0.5 p.p.m. no tastes were produced by phenol with concentrations as high as 1.0 p.p.m. even after standing five days.   (2) The time interval between the application of ammonia and chlorine seemed of little consequence, if the ammonia and chlorine were well dispersed in the water.   (3) In testing for the apparent residual chlorine, the time period required for a maximum reading with ortho-tolidine ranged from one half to one hour.

After conducting this laboratory work, experiments were started on the experimental plant scale, where the rates of flow were varied from 25,000 to 100,000 gallons of water daily.   C.P. Phenol and ammonia-still wastes were used in the water and treated with ammonia and chlorine.   The phenol applied ranged from 0.001 to 1.1 p.p.m. and comparative tests were run with and without the application of ammonia.   The concentrations of ammonia and chlorine used were 0.25 and 0.35 p.p.m., respectively.   This work confirmed the work done in the laboratory.

Work was next conducted on sterilization.   The rates of flow varied from 25,000 to 300,000 gallons daily.   The first tests were with prepared bacterial suspensions obtained by selective breeding; then heavier pollutions were prepared with spore-formers and virulent types of bacteria, and finally raw sewage was applied with a B. coli index of 1,000,000 per 100 cc.   All of the above work was conducted with chlorine and chloramine at temperatures of 10° and 37°C. Samples were planted immediately after treatment, 24 hours, 48 hours, and 5 days.   The samples were treated with about 0.4 p.p.m. of chlorine, and duplicate samples with 0.25 p.p.m. ammonia plus the 0.4 p.p.m. chlorine.   Residual chlorines and chloramines were recorded hourly throughout the day for 5 days, and samples planted for both bacterial counts and gas fermentation in lactose broth.

From all the work previously described in this paper, the following conclusions were drawn:

    (a) Phenolic and chlorinous tastes and odors could be effectively prevented.

    (b) Bacterial efficiencies were improved by the use of ammonia in conjunction with chlorine.

(c) Persistent sterilization would improve the distribution system by preventing aftergrowths.

(d) If there were lagging effects in sterilization, this could be compensated by the addition of larger dosages of chlorine without producing tastes.

From the results obtained during the two years of experimental work on activated carbons and ammonia-chlorine process, the City Administration adopted the latter as the most expedient remedy for phenolic tastes which occasionally occur in the City's water supply. The by-product plants of the Cuyahoga Valley had already co-operated with the Water Department in cleaning up the situation and had done a fairly good job, but occasionally tastes did occur.

The reasons for adopting this method were:

(1) The process lends itself readily to our plants for either continuous or intermittent treatment.

(2) Installation, operation and maintenance costs are very reasonable.

(3) Chlorophenolic tastes and odors are prevented.

(4) By maintaining higher and more stable chlorine residuals, the distribution system was maintained in a sterile condition.

(5) It is the most practical and economical method known for phenolic taste prevention.

The Division Filtration Plant started using ammonia November 1, 1929, and applied the ammonia to the water by means of an ammoniator constructed at that plant. The ammonia was applied first as a solution in the effluent lines leaving the filter plant some 900 feet distant from where the water enters the filtered water reservoir and at which point the chlorine is applied. This reservoir has about 6 hours' storage before the water is pumped into the distribution system.

At Baldwin Filtration Plant the ammonia was first applied January 4, 1930, by means of a Pardee Ammoniator. The ammonia is applied as a gas to the four filtered water effluent lines located some 30 to 75 feet prior to the point of application of the chlorine. The filtered water then flows directly into the filtered water reservoir which has practically a day to a day and a half storage before entering the distribution system.

Our distribution system comprises some 1300 miles of piping from 3 to 60 inches in diameter, with two open distribution reservoirs

floating on the lines. This system supplies some 30 suburbs and 16 county sewer districts, covering a total area of 450 square miles, supplying 1,250,000 persons.

After the introduction of the ammonia-chlorine process, a county-wide survey was begun and samples taken weekly over the total area at 39 different sampling points on the east, west and south ends of the county.

TABLE 1

Cost of disinfection for Cleveland water supply

	DIVISION FILTRATION			
	1929		1930	
		per m.g.		per m.g.
Pumpage, m.g.	31,456		31,888	
Cost of chlorine	$5,641.01	$0.179	$5,276.51	$0.165
Operation labor	448.45	0.014	859.62	0.027
Maintenance cost	345.52	0.011	754.31	0.024
Cost of ammonia*	685.80	0.138	6,101.63	0.194
Total cost	$7,120.78	$0.226	$12,992.07	$0.407

	BALDWIN FILTRATION			
	1929		1930	
		per m.g.		per m.g.
Pumpage, m.g.	33,584		33,184	
Cost of chlorine	$6,425.22	$0.191	$4,091.75	$0.123
Operation labor	354.75	0.011	867.43	0.026
Maintenance cost	361.07	0.011	548.16	0.017
Cost of ammonia†			4,777.32	0.144
Total cost	$7,141.04	$0.213	$10,284.66	$0.310
Grand total	$14,261.82	$0.219	$22,276.73	$0.342

* Started use of ammonia at Division, November 1, 1929.

† Started use of ammonia at Baldwin, January 4, 1930.

This survey covered practically all of the year 1930, in which some 1150 samples were collected and about 5,000 chemical and bacteriological tests were completed.

Division Filter Plant, located on the west side of town, maintained a constant feed of 2 pounds NH_3 and 3 to 4 pounds of chlorine for the first half of the year, or through the taste period, after which the ammonia was reduced to one pound per million gallons. Baldwin

Filtration Plant, located on the east side, varied the ammonia from 0.5 to 2 pounds and the chlorine from 2 to 5 pounds per million gallons throughout the year, except on three different occasions for a period of one week when ammonia was discontinued. These weeks were in April, June and October, when the temperature of the water was 6°, 16° and 20°C., respectively.

When ammonia was applied, the residual chlorine was maintained throughout the distribution system and bacterial results were materially improved, which was not the case when the ammonia was discontinued.

Chlorine residuals as high as 0.55 p.p.m. did not produce tastes, nor were chlorophenolic tastes noted during the season when expectant tastes generally occur.

In conclusion, this method of treatment gave the following results:

(1) No chlorophenolic or chlorinous tastes and odors have been noticed since the adoption of the ammonia-chlorine process.

(2) Highly satisfactory sterilization with apparently no delay has been obtained at both plant effluents and throughout the distribution system.

(3) Due to the maintenance of at least 0.1 p.p.m. of chlorine residuals throughout the distribution system, aftergrowths and algae have been entirely eliminated, even after the water passes through the two open reservoirs.

(4) No increase in the amount of chlorine previously applied was necessary when ammonia was added to produce as good a bacterial effect as when chlorine was used alone.

(5) That the total cost of operation and maintenance at both plants for 1930 was 34 cents per million gallons treated, compared with 22 cents per million gallons the previous year when chlorine alone was used. Detailed cost data are shown in table 1.

The City Health Department checks the city water daily from various points on the distribution system, and their records show that back in 1916, when we used chlorine alone for improving our water supply, 49 percent of the samples taken throughout the distribution system showed presumptive gas formers. In 1918, when half of our city water supply was filtered, the number of gas formers dropped to 36 percent. By 1921 we were down to 25 percent and, after starting the Baldwin Filtration Plant in 1925, thereby filtering all of the city

water supply, we reduced the gas formers to between 11 and 14 percent for the years 1926, 1927, 1928 and 1929; but for the year 1930, in which ammonia was used in conjunction with the chlorine, the Health Department had only one sample showing B. coli in 1380 samples collected.

The writer believes that the persistent action of chloramines would be a material aid in relieving bacterial loads on filters and has consequently recommended the adoption of both preammoniation and prechlorination of the raw water for Cleveland.

This precedure could also adapt itself readily to cities having no filtration as an additional safeguard for the consumers by the permissible maintenance of higher chlorine residuals throughout the distribution system.

ACTIVATED CARBON AT BAY CITY'S FILTRATION PLANT

By Louis B. Harrison[1]

The layout of Bay City's filter plant is similar to most alum treatment plants. Two coagulation basins, each of 2 million gallons settling capacity, are located south of the head house, and north of the head house is the filter gallery. The effluent of each filter passing through the controller enters directly into the clear water basins.

Two alum solution tanks are located in the head house where the solution is fed directly into the stilling chamber, from whence the water divides and flows through the hydraulic flumes to the mixing chamber and then to the coagulation basins.

The only point where the water can be met and a definite dose of any chemical added, is the head house. After the water leaves the head house there is no provision for applying additional treatment of any kind. This type of plant structure has this outstanding fault —lack of flexibility. The common practice of adding chlorine at the gate house is a very poor one, where chlorine treatment is regarded as an integral part of the purification process and not just an emergency measure.

In order that a modern purification plant may meet public demands for a pure and good tasting water, plants must be constructed in the most flexible manner so that, when necessary, treatments may be applied to the raw, the settled and the filtered waters. The plant must be ready to treat any type of water irrespective of the nature of the raw water entering it. The objective of the purification plant must be to produce, at all times, a water which will meet the standards for bacteriological safety, will be clear and colorless, of uniform hardness and free from disagreeable tastes and odors. In order to produce this kind of water in the Bay City plant we must resort to the use of ammonia-chlorine, alum, lime and activated carbon. The ammonia-chlorine and alum treatments are continuously in use and the lime and activated carbon are put into service whenever the oc-

[1] Superintendent of Filtration, Bay City, Mich.

casion arises.   This paper is devoted mostly to laboratory and plant results with activated carbon.

The Bay City Filter Plant was put into operation in July, 1925, and from the first day of its operation the taste problem became acute.   Not only one taste bothered us, but tastes of various types, although the chlorphenol type was considered the most objectionable. The successful removal of chlor-phenol tastes with potassium permanganate brought about a great disappointment, for the swampy and musty tastes become more prominent after the removal of the medicinal taste, and, from the consumers' point of view, were considered just as objectionable.   A sudden rise in the hardness and chloride content has caused a change in taste.   The Saginaw Bay is shallow and a change in wind produces a change in the raw water. A southeast wind brings industrial contamination from the Saginaw River and chlor-phenol tastes as well as hard water high in chlorides. A northeast wind brings about the same condition in a milder form. A strong southwester brings marshy tasting water from the Tobico swamp, and a northwester brings lake water which is soft and pleasant tasting.   The plant had to be adjusted so that irrespective of the type of water Providence ordains to enter it, a reasonably uniform type must leave it.

The laboratory and plant results with super-chlorination and dechlorination, potassium permanganate and ammonia-chlorine have demonstrated that these treatments have only specific applications to medicinal tastes; ammonia-chlorine as a preventor of their formation and permanganate as a destroyer.

In June, 1929, when Baylis[2] published his paper, "The Activated Carbons and Their Use in Removing Objectionable Tastes and Odors from Water," I procured a quantity of Hydro-Darco from the International Filter Company and started experiments with it.   The experimental filter used is the kind Mr. Baylis has been using, except that a constant head device has been incorporated so that the water level over the Hydro-Darco could be kept as constant as possible. This device consists of a $\frac{1}{4}$-inch ball type check valve and $\frac{1}{4}$-inch pet cock.   A $\frac{1}{8}$-inch hole has been drilled through the plug of the check valve, a rigid rod put through it and fastened to the ball.   To the outer end of the rod a cork of the proper size to fit the diameter of the Pyrex glass tube has been fastened.   This check valve has been incorporated in the siphon arrangement and the water admitted to the

[2] This JOURNAL, June, 1929, page 787.

unit by means of a rubber tube connected to the $\frac{1}{4}$-inch pet cock. The length of the rod has been adjusted to meet the required head over the filtering material. As the water flows into the filter the cork rises and falls and automatically controls the flow of the water. The Hydro-Darco was supported by several inches of graded gravel. Twenty-four inches of Darco were used and the water passed through it downward at the rate of 2 gallons per square foot.

Experiments have been conducted on the various types of tastes in the raw water, generally caused by gas plant wastes, wood distillation wastes, swamp drainage, with gratifying results. When a water is very high in its organic content it is advisable to remove a great deal of the organic material before passing it through the carbon unit. To accomplish this, potassium permanganate, super-chlorination or lime with or without soda ash may be used, depending on the character and hardness of the water. The results are much better if the water is thus treated, and the load on the carbon unit greatly reduced. After months of successful laboratory experimentation I became satisfied that carbon units will remove the tastes occurring in our water supply and that the results justify a large scale plant installation.

The chlor-phenol tastes are prevented by the use of the ammonia-chlorine treatment and hence are at present of secondary importance. However, in order to use this treatment with good results, it became necessary to insist that the industries keep out of the stream, not only excessive amounts of phenols, but also free chlorine. The ammonia-chlorine treatment will prevent the formation of chlor-phenol tastes, but will not destroy it, a fact well established by laboratory and plant experiments. The carbon unit is, therefore, used to remove other organic tastes.

The effluent of the carbon unit is rather flat in taste and in order to overcome this I tried a mixture of one part filtered water to five parts of carbon treated water, and this blending resulted in a good tasting water. An absolutely tasteless water is not the most palatable water. Whether it will be possible to get as good a mixture on a plant scale, however, remains to be seen.

PLANT INSTALLATION AND RESULTS

It took considerable time to figure out the best way of converting one of the sand filters into a carbon unit. I may say that Bay City is fortunate in this respect since it has ample filtering capacity. The

average water consumption is 5 m.g.d. and the total plant capacity 20 m.g.d. Our peak loads reach 15 m.g.d. on certain days during the summer months. The 5 m.g. storage clear water basin takes care, however, of these fluctuations. It is obvious that reducing the plant capacity to 16 m.g.d. does not in any manner inconvenience the plant. At present, I believe, Bay City has the largest carbon unit installation.

The sand was as far as possible removed and replaced with 1,400 cubic feet of Hydro-Darco which made a 24-inch bed. Special fittings were connected to the outlet of the rate controller of the adjacent sand filter so that the effluent could either be discharged into the clear water basin or by shutting a valve, diverted to the carbon bed. A De Laval, 10-inch suction, 10-inch discharge, single stage, double suction, horizontally split case centrifugal pump connected to a 5 H. P. motor was installed to pump the filter effluent to the carbon unit and raise it to the same level as the water over the sand filters.

The controller on the carbon unit and on the filter bed are set as far as possible at the same rate. In order to put the unit into operation a 16-inch valve is shut and a 10-inch valve opened, the pump started and the effluent valve opened on the carbon unit. I have experienced some trouble at first in washing the carbon unit, for it did not take long to find out that the high velocity wash rate used on the sand filter will cause a great loss of carbon. It has been necessary to reduce the wash water rise from 25 inches to a 12-inch rise. The carbon must be washed free from hydrochloric acid in order to prevent damage to the metal troughs and pipe lines in the unit.

The flow through the unit is downward, and in order that the filter attendant may watch the water level in the carbon unit a float gage with a board marked off in fractions of an inch has been placed over the carbon unit. During actual operation it has been found that the rate of filtration hardly fluctuated until the loss of head gage in the sand filter registered 4.5 feet. Then a slight readjustment of the weight on rate controller lever kept the rate constant until the final loss of head of 9 feet was reached.

The carbon unit has been in actual operation for six days during the fall of 1930. The results were highly pleasing and in spite of the fact that only 2 million gallons of water per day have passed through the carbon unit and mixed with 4 million gallons untreated water, not a single complaint has been received.

No process for reactivating the carbon has been installed, nor do

I know when it will be necessary to install it. We have steam on hand and in the future we may make steam connections to the manifold, so that steam can be passed through the bed in the same manner as the wash water. I am, however, marking time with this installation, waiting for developments in the field of reactivation.

The total cost of the installation amounted to $5,600.00, and an additional expenditure of a similar amount will, I hope, completely solve for some time the taste problem in Bay City's water supply.

EXPERIENCE WITH POWDERED ACTIVATED CARBON

By Malcolm Pirnie[1]

The availability of a cheap powdered activated carbon opens wide the field for experimentation with its use in combating tastes and odors in water supplies from existing water purification plants. Several plants have adopted the application of powdered activated carbon to remove tastes and odors, some shifting the point of application in a search for the most effective treatment with existing facilities. None, however, has yet made structural changes in facilities designed to secure the greatest efficiency of the carbon.

Through the coöperation of those responsible for introducing powdered activated carbon in the treatment of water at fifteen purification plants, it has been possible to summarize the methods used and results obtained. In this early experimental stage, with a substance that acts differently on different impurities in water, it is not surprising to find differences in adopted methods and in results obtained from its use in the treatment of water from fifteen different sources of supply.

The popular method of applying the powder is to run it through a dry feed machine into a hopper where water is added and a water eductor picks up the mixture and forces it through a pipe or hose to the point of application. Efficient distribution through the water appears highly desirable. At one small plant a pound of powder is thrown into the filter wash water trough when influent water is admitted following the washing operation. The efficacy of this method is naturally limited. Another way is to maintain a water suspension of the powder by agitation and pump the mixture through a pipe fitted with nozzles located below the water surface at the point of application. Apparently the dry feed and water eductor means of introducing proportional amounts of the powder are best adapted for feeding considerable volumes and present no difficulties. The black dust makes it desirable to handle the powder in a room or building separated from other operating rooms of the plant and it is

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[1] Consulting Engineer, New York, N. Y.

well to employ all available precaution to avoid creation of dust during handling and feeding.

Raw water pump suctions, coagulation basin mixing chambers, intermediate points in the coagulation basins, effluent conduits from coagulation basins to filters and to the water on top of filters have each been employed, for one or another reason, as points of application of powdered activated carbon. In one of the fifteen plants the carbon is added to the water on top of the filters, in two plants it is added to the effluent conduits from coagulation basins and in twelve plants it is added either to the raw water or water in the mixing chambers close to the point of alum treatment. The three plants adding carbon just prior to filtration secure satisfactory results with doses of less than 2 p.p.m. With one exception larger doses were found necessary in the other plants which resulted in such rapid clogging of the filters that the points of application were shifted to the raw water or the coagulated water entering the coagulation basins. The exception above referred to is the Iowa City plant applying 0.7 p.p.m. of carbon close to the alum dosing point. From reports received the effect on filter runs of introducing the carbon in the applied water appears to be about as follows:

CARBON DOSE	EFFECT ON FILTER RUNS
p.p.m.	
17.0	Reduced to three hours
8.5	Reduced 50 percent
2.4	Clogs filters rapidly
1.2–1.7	Satisfactory
0.25–0.30	Satisfactory

In instances the choking of filter beds has been overcome by employing the "bed jolting" scheme of opening the wash valve to raise the bed at intervals without actually washing the filters.

Patrick at Luke, Maryland, reports experiments with a powdered activated carbon known as Nuchar No. 2. Adding 50 p.p.m. Nuchar to a sample of black water which is waste from the de-inking process in the preparation of old newspapers for use as paper pulp, and allowing the mixture three hours' contact with occasional shaking, he found that Nuchar caused a 30 percent reduction in dissolved oxygen and a 61 percent reduction in the biological oxygen demand. The black water before treatment had a pH 4.4, dissolved oxygen

8.0 and b.o.d. 59.1. He also tried the effect of adding Nuchar to distilled water containing sufficient chlorine to give a residual of 0.5 p.p.m. With 5 p.p.m. of Nuchar, five hours was required to reduce the residual chlorine to 0.1 p.p.m. Using twice as much Nuchar, however (10 p.p.m.) reduced the residual chlorine to 0.1 p.p.m. in half an hour or one-tenth the time required for the 5 p.p.m. dose. When added to a water containing free CO_2 Nuchar has caused an increase in pH by adsorbing this gas. There is an interesting possibility that given quantities of activated carbon may remove impurities causing a definite taste at one pH and be relatively ineffective at another pH. Carbon contact with raw water before alum application, after alum application or with water that has been treated with soda or lime presents varying pH conditions for the exercise of its adsorptive properties.

Apparently without structural changes in existing filter plants the most intimate contact between powdered activated carbon and the water can be secured by adding the carbon to the applied water leaving the coagulation basins or at some point convenient enroute to the filters. All the carbon is thereby kept in contact with the water for a period averaging, roughly, half the length of a filter run. If 2 p.p.m. or less applied in this manner will remove the tastes and odors, it will prove to be the least expensive method. Larger doses may be found practical if the clogged surface of the filter sand is broken—"jolted"—periodically by closing the effluent valve and opening and closing the wash valve. When materially larger doses of carbon are necessary, applying it to raw water pump suctions or supply lines or to water in coagulation basin mixing chambers has been effective, but more carbon is undoubtedly needed than would be the case if applied to settled water after coagulation.

Reports come from plants treating some of the most highly polluted waters used as public water supplies, from others treating moderately polluted waters and from some treating waters with little or no pollution, but having tastes due to microörganisms. One plant reports 240 p.p.m. powdered carbon added in the mixing chamber would be necessary to produce satisfactory taste removal; other plants get satisfactory results with 17 to 25 p.p.m. added to the raw water, and two plants have eliminated tastes and odors with from 0.25 to 1.7 p.p.m. at costs varying from 11 to 75 cents per million gallons treated.

Tastes and odors due to sulphite liquor, aldehol, phenols and micro-

organisms have been completely eliminated with the aid of powdered activated carbon in several plants. In other plants treating extremely polluted raw waters satisfactory reduction in tastes and odors has not yet been secured with the powdered carbon. Experiments at two of these latter plants have shown that passage of *the filtered* water through 30 inch beds of granular Hydro Darco will render the water tasteless.

There is obviously a large field for use of powdered activated carbon in existing water purification plants and particularly in those plants giving normally tasteless effluents, but troubled with occasional tastes and odors. Plants treating moderately clean waters can in many cases secure taste removal by introducing small doses of carbon just prior to filtration. Other plants with long raw water supply lines may find application of carbon to the intakes most beneficial or satisfactory taste removal may result from carbon application in the coagulation basin mixing chamber. Some plants treating waters polluted with wastes highly resistant to the treatment employed, if equipped with adequate sedimentation capacity prior to filtration, may find it possible to apply powdered carbon to partly settled water leaving basin capacity for sufficient settlement of the carbon before the water enters the filters. Removal of objectionable tastes and odors is certainly worth the moderate cost indicated by the majority of those who have achieved this with introduction of powdered activated carbon.

DISCUSSION

F. E. STUART:[2] I shall give you a résumé of some of the various methods of Nuchar application, together with cost and results in plants located in various sections of the country.

Hamburg, N. Y., Mr. E. A. Stern, Superintendent.   The method used is a dry feed machine equipped with a high speed rotary pump with spray nozzle connection.   Solution is applied to the filter influent line.   The amount used is 0.035 g.p.g. and the per million gallon cost is 30 cents.   The results from this treatment are satisfactory and with no increased wash water consumption.   Prechlorination is practiced.   The supply is from a very muddy creek.

New Castle, Pa., Mr. B. F. Johnson, Chemist.   The method used is a dry feed machine with a water ejector.   A solution is applied to the coagulated water basin flowing to the filters.   The amount used is 0.195 g.p.g. and the per million gallon cost is $1.35.   The results from this treatment have been entirely satisfactory with complete taste and odor removal.   Pre-chlorination is practiced at times. River supply.

Olean, N. Y., Mr. S. D. Poarch, Superintendent.   The method used is a dry feed machine with ejector solution to outlet of settled water basin, just previous to the filters.   The amount used is 0.045 g.p.g. and the per million gallon cost is 39 cents.   The results from this treatment are complete removal of pre-chlorination taste; also removal of dead algae taste resulting from pre-chlorination.   The supply is from a small river, heavily polluted.

Washington, Pa., Mr. Newquist, Chemist.   The method used is a cut-and-try method, carbon applied to the filters by hand and claims of no increase in wash water are made.   The amount used is 0.09 g.p.g. and the per million gallon cost is 78 cents.   The results from this treatment are complete removal of taste from high copper sulphate treatment and excessive pre-chlorination.   These results are very encouraging as the copper sulphate produces many dead microscopic organisms that in turn have caused considerable taste annoyance in the past.   The supply is from impounding reservoirs.

Morgantown, W. Va., Mr. Charles Cook, Superintendent of Filtration.   The method used is very peculiar, as he is mixing the carbon with lime in about a ten to one proportion and applying with a dry feed machine to the coagulated water basin.   The amount used is 0.10 g.p.g. and the per million gallon cost is 70 cents.   The results

[2] Water Purification Division, Industrial Chemical Sales Company, New York, N. Y.

from this treatment are complete taste and odor removal. The supply is from a river.

Powdered activated carbon has also entered the field of water softening.

Daytona Beach, Fla., Mr. B. Tippins, Superintendent. The method used is another peculiar one, as the carbon is applied to the alum solution tanks and fed to the water entering the down-take in the softener, the effluent of which passes to pressure filters. The cost is $1.40 per million gallons and gives excellent results in removing the characteristic flat taste from the heavily softened water by the lime-soda process. Sparkling effluent with a good taste even at high temperatures results. The supply is from artesian wells. This incidentally is the first softening plant in the country to use activated carbon to remove the characteristic tastes that result in water softening plants that do not have recarbonation units to give them a balanced water. A survey is being made for future installations.

Activated carbon in the powdered form seems to be applicable in either an electropositive or an electronegative solution and does not interfere with previous methods of treatment, leaning favorably to the electronegative side.

The results and costs vary with the conditions under which these plants were operating at the time of the writer's visit. It is understood that these cost figures will vary with the local conditions at different seasons.

To date we have been successful in removing the objectionable tastes and odors with which we have come in contact. From my observations of this method of treatment, the ideal treatment will be in most instances as follows:

Pre-chlorination should be practiced in order to derive the several noticeable advantages; powdered activated carbon should be applied to the filter influent water to remove the excess chlorine or any taste resulting from pre-chlorination. After filtration, ammonia should be added in a small concentration to assure the residual chlorine content far out in the distribution system. Of course, this takes into consideration the fact that chlorine will be the final chemical added to the filtered water.

In some cases where conditions may be extremely severe, pre-chlorination may have to give way to post-chlorination, the carbon being applied at one or more points ahead of the filters.

With this method of treatment, I believe there is no water that cannot be successfully purified, if the proper application of the aforementioned chemicals is made.

THE OCCURRENCE OF FLUORIDES IN SOME WATERS OF THE UNITED STATES[1]

By H. V. Churchill[2]

Investigators of the dental defect known as mottled enamel seem to agree on two points: the defect occurs in certain geographical areas, and the causal factors seem to be associated with the water supply of those areas. Reference to the complete bibliography[3] on the subject compiled by Kempf and McKay show that no specific common characteristic of the waters from affected areas has been discovered. Following the publication referred to above, A. W. Petrey of this laboratory spectrographically discovered the presence of fluorides in deep well water from Bauxite, Arkansas. These deep wells were formerly the source of water used by individuals who show the characteristic dental defect known as mottled enamel. Several months prior to the investigation covered in the report of Kempf and McKay the deep well supply at Bauxite was abandoned in favor of another supply which has since been found to be free from fluorides.

The presence of fluorides was first revealed by means of the spectrograph. Substances containing both calcium and fluoride show a characteristic spectral band having its head at 5291A and which is degraded towards the infra red end of the spectrum. The spectrum obtained from Bauxite deep well water is shown in figure 1. The definite and specific character of the 5291A band of calcium fluoride is shown in figure 2 which shows the spectra obtainable from a variety of calcium salts. Only those salts which contain both calcium and fluoride show the characteristic band indicated by the limiting lines in the cases of the spectra of calcium fluoride and that of a mixture of calcium oxide and aluminum fluoride.

Following the discovery of fluorides in Bauxite deep well water, the author secured samples of water from other localities where the defect occurs. These localities were Colorado Springs, Colorado, a well near Kidder, South Dakota, a well near Lidgerwood, North

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[1] Published by the courtesy of Industrial and Engineering Chemistry.

[2] Aluminum Research Laboratories, New Kensington, Pa.

[3] Public Health Reports, 45: 48, November 28, 1930.

Fe Arc.
Bauxite, Ark.

Fig. 1. Spectrum of Deep Well Water

Fe
Ca-Metal
CaO
CaCO₃
CaCl₂
CaSO₄
CaF₂
CaO + AlF₃
AlF₃
Electrode

Fig. 2. Spectra Obtainable from a Variety of Calcium Salts

Dakota, and Oakley, Idaho.   Spectrograms of the water from these localities are shown in figures 3 and 4.   The presence of fluorides is definitely shown in all cases.

When the residues from these waters after evaporation were tested qualitatively for fluorides by etching methods[4] positive tests resulted in all cases.   While the etching produced in some cases is slight, nevertheless in all cases it was demonstrably positive.

Quantitative estimation of fluorine is fraught with difficulty.   In our opinion results for fluorine even with the best available methods tend to give low results.   For the determination of fluorine in the samples herein considered recourse was taken to a method of Fairchild.[5]   Since this method was worked out for particular use on phosphate rock, it was necessary to adapt it to the analysis of water. The method, as finally used in the work covered in this paper, was as follows:

Pipette 100 cc. of water in a 250 cc. glass stoppered Erlenmeyer flask, add 1 drop methyl red, and 1.0 N hydrochloric acid dropwise until acid.   Add 10 cc. 20 percent sodium chloride solution.   Filter, wash and discard any precipitate.   Add an excess of 0.08 M ferric chloride over that required for completion of the reaction,

$$FeCl_3 + 3NaF = FeF_3 + 3NaCl$$

Five cubic centimeters is the proper amount for fluorine content from 0.0001 to 0.0100 gram.   Add 2 cc. 1.0 N hydrochloric acid and 10 cc. 5 percent potassium iodide solution, and stopper the flask.   Immerse it in a water bath at a temperature of 38 ± 1°C. and allow it to remain in the bath thirty minutes.   Iodine is liberated by the ferric chloride in excess of that required to react with the fluorine present. Quickly cool the flask, and titrate the iodine with standard thiosulfate solution, using starch as indicator.   Simultaneously carry through a control sample with 100 cc. of distilled water and the quantities of reagents used in the analysis.   The difference in thiosulfate consumption between the control and the sample represents the ferric chloride consumed in the reaction with fluoride.   If the thiosulfate solution contains 4.354 grams of the crystallized salt per liter, it will be equivalent to 0.001 gram fluorine per cubic centimeter.

If the fluorine content shown by the analysis is very low, repeat the analysis, with a larger sample.   Evaporate this to 100 cc. in platinum.

[4] Scott, Standard Methods of Chemical Analysis, 1: page 212.
[5] Journal of The Washington Academy of Sciences, 20: 8, page 141.

Fig. 3. Spectrogram of Spring Water

Fig. 4. Spectrograms of Various Waters

Scott, Standard Methods of Chemical Analysis, 1st page 378.

Journal of The Washington Academy of Sciences, 20: 6, page 144.

Do not allow the solution to become strongly concentrated during the evaporation, lest hydrofluoric acid be lost.

Application of this method to various samples from endemic areas yielded the following results.

SAMPLE	FLUORINE AS FLUORIDE
	p.p.m.
Bauxite deep well...	13.7
Colorado Springs, Colo......................................	2.0
Well near Kidder, S. D......................................	12.0
Well near Lidgerwood, N. D.................................	11.0
Oakley, Idaho...	6.0

It is well to emphasize the fact that no precise correlation between the fluoride content of these waters and the mottled enamel has been established. All that is shown is the presence of an hitherto unsuspected common constituent of the waters from endemic areas.

TABLE 1

List of public water supplies in which fluorides are present

Cincinnati, Ohio	Detroit, Mich.
Milwaukee, Wis.	Peoria, Ill.
Birmingham, Ala.	Indianapolis, Ind.
Pittsburgh, Pa.	Buffalo, N. Y.
St. Louis, Mo.	Davenport, Iowa
Kansas City, Mo.	San Francisco, Cal.
Toledo, Ohio	Minneapolis, Minn.
Cleveland, Ohio	Los Angeles, Cal.

TABLE 2

List of public water supplies in which fluorides are not present

Philadelphia, Pa.	Boston, Mass.
Washington, D. C.	Newark, N. J.
Albany, N. Y.	Chicago, Ill.
Fairfield, Conn.	Dallas, Texas
New York, N. Y.	Atlanta, Ga.

However, it is of interest to note that apparently the relative severity of the defect in these various areas seems to follow the fluoride concentration.

Since the occurrence of fluorides in potable waters has apparently
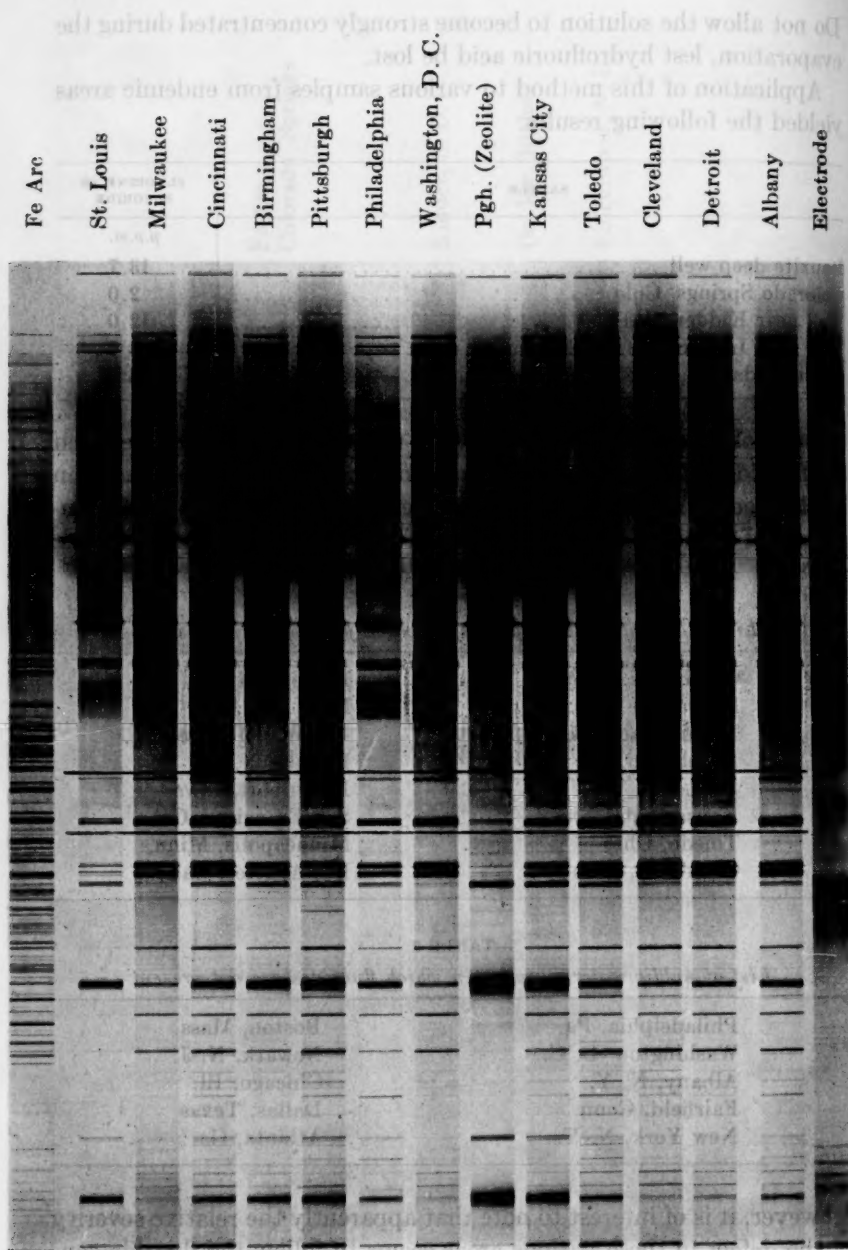
Do not allow the solution to become strongly concentrated during the evaporation, lest hydrofluoric acid be lost.

Application of this method to various samples from endemic areas yielded the following result:

Fe Arc
St. Louis
Milwaukee
Cincinnati
Birmingham
Pittsburgh
Philadelphia
Washington, D. C.
Pgh. (Zeolite)
Kansas City
Toledo
Cleveland
Detroit
Albany
Electrode

Fig. 5. Spectrograms of Various Waters

However, it is of interest to note that apparently the spectrograms show the natural fluorine content alone. In follow up studies concentration.

Since the occurrence of fluorides in potable waters has apparently

FIG. 6. SPECTROGRAMS OF VARIOUS WATERS
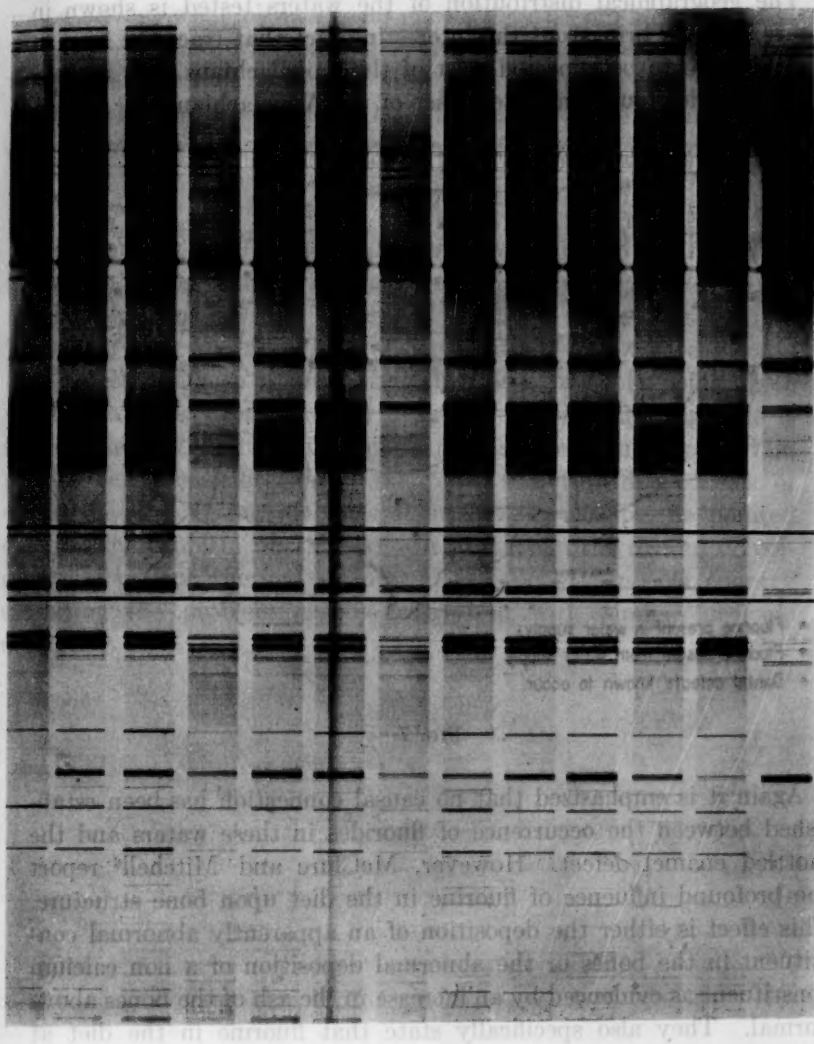
not been widely studied, a survey of some municipal supplies in the United States was undertaken with the results as shown in tables 1 and 2.

Figures 5 and 6 show spectrograms of the various waters. Quantitative examination of the waters covered in tables 1 and 2 shows the fluorine content in all cases to be less than 1.0 p.p.m.

The geographical distribution of the waters tested is shown in figure 7 which brings out the interesting fact that traces at least of fluorides are to be expected west of the Appalachians. In no case were fluorides found in waters east of the Appalachians.

■ Fluorine present in water supply.
● Fluorine absent from water supply.
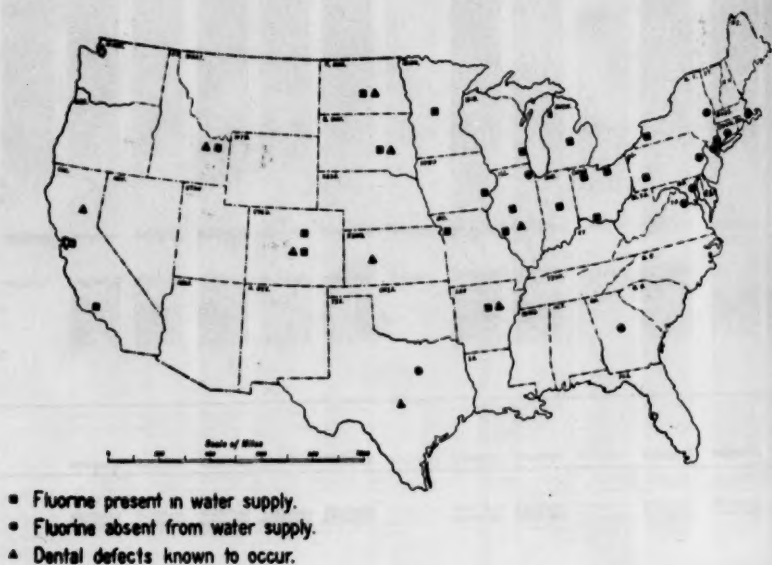▲ Dental defects known to occur.

FIG. 7

Again it is emphasized that no causal connection has been established between the occurrence of fluorides in these waters and the mottled enamel defect. However, McClure and Mitchell[6] report the profound influence of fluorine in the diet upon bone structure. This effect is either the deposition of an apparently abnormal constituent in the bones or the abnormal deposition of a non calcium constituent as evidenced by an increase in the ash of the bones above normal. They also specifically state that fluorine in the diet at

[6] Journal Biological Chemistry, 90: 297–320, 1931.

certain levels has a peculiar effect upon the development of teeth. Pending establishment of causal connection between fluorine at certain concentrations in water and the mottled enamel defect, water chemists might well give attention to the problem of the control of fluoride concentration in drinking water. Two questions are raised by this discovery of unsuspected amounts of fluorine in drinking water. First, what physiological effects may be produced by these fluorides? Second, what can water chemistry contribute to the concentration control of fluorides?

Acknowledgment is given to Dr. F. S. McKay of the Public Health Service for help in securing water samples from endemic areas.

DISCUSSION

MR. GOUDEY (Hollywood, Cal.) called attention to the fact that, if fluorine is absent from the food or water, growing children will not be furnished all the materials necessary to tooth structure. Thus it would seem, in the light of suggestions by Mr. Churchill as to over concentration of fluorine, that there is a threshold point up to which fluorine concentration is desirable and beyond which it is undesirable. Mr. Churchill replied that the problem appeared in past to be a decalcification of the tooth structure due to lack of dietary balance as regards mineral intake. In closing, he pointed out that the problem was first known in Italy in the regions of volcanic activity or thermal springs. Each locality in this country exhibiting the problem can be correlated with similar geologic conditions.

ABSTRACTS OF WATER WORKS LITERATURE[1]

FRANK HANNAN

"The Economizer"—Some Recent Developments, With Experimental Data and Conclusions. G. E. TANSLEY. Proc. S. Wales Inst. Eng., 45: 480–512, 512–19, 1930. From Chem. Abst., 24: 1769, April 20, 1930. Water can be heated more cheaply and efficiently in an economiser than in a boiler. The function of the latter is simply to supply latent heat. The greater the steam pressure the greater should be the proportion of economiser surface and the less the boiler surface. The modern economiser differs from the older machine in its enormously greater gas velocity. Experimental data are given on the most efficient ratio of external to internal surface for economiser tubes. A fin 1 inch high and thick at the root (for 3½-inch pipe; surface about 3 square feet to the foot) gave the best service of the types of tubes tested. The vertical type of economiser is easier to clean than the horizontal type.—*R. E. Thompson.*

Purity of Lead at the Time of the Birth of Christ. AUGUST EBELING and HANS ADAM. Wiss. Veröffentlich. Siemens-Konzern, 8: 3, 203–10, 1930. From Chem. Abst., 24: 1774, April 20, 1930. Lead from graves on the Dalmatian coast of a period prior to the birth of Christ and a section of water pipe from Pompeii were analyzed. The earlier lead was more pure, the samples containing 99.8 and 99.3 per cent lead respectively. The cause of this difference could not be established.—*R. E. Thompson.*

The Iodine Content of the Ocean. J. F. REITH. Rev. trav. chim., 49: 142–50, 1930. From Chem. Abst., 24: 1817, April 20, 1930. A summary is given of the methods that have been proposed for determining iodide in the presence of considerable chloride. The 2 most important methods were tested. Good results were obtained with the WINKLER method. The alcohol extraction method, on the other hand, gives satisfactory results only when the final titration is made in acetic acid solution. Probably many determinations reported in the literature are wrong for this reason. About 0.043 p.p.m. iodine was present in all ocean water tested.—*R. E. Thompson.*

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[1] Vacancies on the abstracting staff occur from time to time. Members desirous of coöperating in this work are earnestly requested to communicate with the chief abstractor, Frank Hannan, 285 Willow Avenue, Toronto 8, Ontario, Canada.

The Problem of Dilution in Colorimetric Hydrogen-Ion Measurements. II. Use of Isohydric Indicators and Superpure Water for Accurate Measurement of Hydrogen-Ion Concentrations and Salt Errors. S. F. ACREE and EDNA H. FAWCETT. Ind. Eng. Chem., Anal. Ed. 2: 78–85, 1930. From Chem. Abst., 24: 1567, April 10, 1930. The production, testing, and storage of water of pH 7 are described. The principal difficulty with ordinary distilled water is the carbon dioxide content. The need of "neutral" water for colorimetric H-ion determinations in strongly buffered solutions is pointed out. A method is given for adjusting the pH of indicator standards against 0.001–0.05 M buffer color standards; the technic for using the isohydric indicator method is described.—R. E. Thompson.

Microchemical Determination of Iodide in the Presence of Other Salts. J. F. REITH. Biochem. Z., 216: 249–68, 1929; cf. C. A., 23: 3182; 24: 313. From Chem. Abst., 24: 1596, April 10, 1930. An exhaustive comparative study of the various microchemical methods for the determination of iodine. The GRANGE-FRESENIUS titration is satisfactory only with quantities above 20γ iodine, while the WINKLER determination, as well as the VON FELLENBERG colorimetric method, is influenced by the presence of various salts. Sufficiently accurate results are obtained when oxidation with bromine water is employed (instead of chlorine water) in the WINKLER determination either in a sulfuric acid medium or with sea water in an acetic acid medium. It is necessary, however, to eliminate nitrites by means of sodium azide. Quantities of iodine less than 1γ can best be measured colorimetrically—R. E. Thompson.

Microchemical Determination of Iodine. GULBRAND LUNDE, KARL CLOSS and JENS BÖE. Mikrochemie, Special vol. commemorating FRITZ PREGL, 272–92, 1929. From Chem. Abst., 24: 1596, April 10, 1930. A discussion of the proper technic for determining small quantities of iodine in organic materials with special reference to the necessary precautions in igniting the material, enrichment of the iodine, effect of large quantities of calcium salts, and titration of small quantities of iodine.—R. E. Thompson.

Chlorometric, Bromometric and Iodometric Determination of Available Chlorine in Hypochlorites and in Bleaching Powder. I. HUGO DITZ and RUDOLF MAY. Z. anal. Chem., 79: 333–45, 1930. From Chem. Abst., 24: 1596, April 10, 1930. A thorough study of the various methods was made during 1922–5 and in this paper the results obtained with the chlorometric methods of PENOT-LUNGE are compared with those obtained with the iodometric method of RUPP, the chlorometric method of KOLTHOFF and the modification of the PENOT method recommended by RODT. The PENOT-LUNGE method gave results a little lower than the iodometric method, chiefly because the indicator is not quite as sensitive toward available chlorine, and the KOLTHOFF method gave values which were less consistent and appeared to be a little too high. The RODT method gave results which agreed fairly well with those obtained by the iodometric method of RUPP.—R. E. Thompson.

Determination of Alkalies in Waters and in Silicates by a Semi-Microchemical Method. K. L. MALYAROV. Mikrochemie, 8: 63–71, 1930. From Chem.

Abst., **24**: 1600, April 10, 1930. An application of the BERZELIUS method is described for the determination of alkalies in waters and in silicates. The silicate is decomposed by hydrofluoric and sulfuric acids, the excess acid volatilized, and all bases except alkalies removed by precipitation with barium hydroxide. Excess barium is removed as carbonate; ammonium salts, by volatilization; residue is titrated. Calculation is based on the assumption that only sodium and potassium hydroxides are present at the last.—*R. E. Thompson.*

The Carbon Dioxide Content of Distilled Water. WALTER HOESCH. Chem.-Ztg., **53**: 956, 1929. From Chem. Abst., **24**: 1599, April 10, 1930. Distilled water kept in a cellar gave acid values in analyses different from those of the supply from upper floors of same building. In very accurate work the carbon dioxide content of the distilled water should be known, or its influence guarded against.—*R. E. Thompson.*

The Histochemical Detection of Iodine. M. STEINER. Biochemie, **1929**: 1, Part 2, 242–67; Wasser u. Abwasser, **26**: 111; cf. C. A., **22**: 980, 4420. From Chem. Abst., **24**: 1818, April 20, 1930. The iodine cycle in nature and the geochemistry of iodine are discussed. The methods of determination are reviewed. In order to overcome difficulties in the FELLENBERG method, the author proposes combustion in a quartz tube and removal of traces of nitrites previous to the final titration.—*R. E. Thompson.*

Iodine as a Bioelement. I. CARL OPPENHEIMER. Chem.-Ztg., **53**: 925–7, 1929. From Chem. Abst., **24**: 1876, April 20, 1930. Reviewing the studies of various investigators the author finds no explanation for the fact that goiter attacks only part of the people in a region. Complete data on iodine occurrence and on the amount eliminated from the animal body are needed. II. Ibid., 968–9. Initial experimentation leads one to conclude that iodine in fertilizers produces the best hygienic results for plants and animals.—*R. E. Thompson.*

Chlorometric, Bromometric, and Iodometric Determination of Available Chlorine in Hypochlorites (Bleach Liquors) and in Chloride of Lime. II. HUGO DITZ and RUDOLF MAY. Z. anal. Chem., **79**: 371–88, 1930. From Chem. Abst., **24**: 1820, April 20, 1930. On the assumption that RUPP's iodometric method gives correct results, the following conclusions are drawn from the results of considerable analytical work. (1) The method of PENOT in the form recommended by LUNGE gives results which are a trifle low. Error is greatest with chloride of lime containing considerable insoluble matter and may amount to 0.4 percent. The spot test with iodo-starch can be obtained with 0.004 mg. chlorine per cc. (2) The modification of PENOT's method proposed by RODT gives values a little nearer the standard and is about equally sensitive. (3) The PENOT-MOHR method gives practically the same values as the standard. (4) BECKURTS's method is not to be recommended. (5) CLARENS's method gives results slightly higher than PENOT's. (6) KOLT-HOFF's procedure gives values that are slightly higher than the iodometric

method, partly because of the deposition of insoluble material in the buret.—
R. E. Thompson.

Some Very Sensitive Tests with Apomorphine. F. PAVELKA. Mikrochemie,
8: 46–52, 1930. From Chem. Abst., 24: 1600, April 10, 1930. To test for
nitrite, add 2 or 3 drops of 50 percent acetic acid to the dilute solution and 1 or
2 milligrams of apomorphine; heat nearly to boiling, allow to cool, and shake
with ethyl acetate. A violet coloration is obtained when the nitrite concen-
tration is 1:15,000,000. In testing drinking water, use a few drops of dilute
phosphoric acid instead of acetic acid. The test succeeds in presence of
nitrate. A test for both nitrite and nitrate can be made by first testing for
nitrite, and in another portion removing all nitrite by reduction to nitrogen
with sodium nitride and then reducing the nitrate to nitrite with zinc and
testing with apomorphine.—R. E. Thompson.

Evaporation in the Chemical Analysis of Mineral Water. PIERRE BROTEAU.
Bull. acad. méd., (3), 102: 216–8, 1929. From Chem. Abst., 24: 1688, April 10,
1930. The addition of hydrochloric acid cannot always be recommended when
large quantities are to be evaporated. Small quantities of platinum may be
dissolved from the dish and antimony, bismuth, and zinc may escape detection.
—R. E. Thompson.

The Chemistry of Drinking Water. L. DESCROIX. L'eau, 22: 122. 1929.
From Chem. Abst., 24: 1688, April 10, 1930. A plea is made for the cleaning
up of water supplies in conjunction with chlorination. Polluted waters, even
when sterilized with hypochlorite or chlorine, may cause intestinal or other
body disturbances.—R. E. Thompson.

Lime-Aggressive and Rust-Inhibiting Carbonic Acid in Natural Water. J.
TILLMANS. Z. Untersuch. Lebensm., 58: 33–52, 1929. From Chem. Abst., 24:
1688, April 10, 1930. A review of the author's work with theoretical considera-
tions. The varying rôles of carbon dioxide and of oxygen dissolved in water
in relation to the action of natural waters on calcareous and iron structures are
fully considered.—R. E. Thompson.

Reaction Between Soils and Metallic Iron. H. D. HOLLER. J. Wash.
Acad. Sci., 19: 371–8, 1929. From Chem. Abst., 24: 1608, April 10, 1930.
Pulverized cast iron was mixed with the soil to be tested. The mix was poured
into the distilled water, inverted into a beaker full of water and the hydro-
gen evolved after a given time measured. The quantity of hydrogen evolved
was not directly dependent upon pH. Silts and clays, whose buffering action
tends to maintain a constant pH, with low pH gave high evolution rates,
whereas sandy soils of low pH did not necessarily give high rates. The
proportion of colloidal matter in the soils gave indications of the available
supply of H ions. Since with few exceptions the gas evolution was nearly
a straight line function of total acidity, the latter was concluded to be the
probable cause of soil activity and of hydrogen evolution.—R. E. Thompson.

Corrosion of Cast Iron in the Ground. A. ROMWALTER. Mitt. berg-hütten-mann. Abt. kgl. Hochschule Berg-Forstwesen Sopron. Ungarn 1929, 217–24. From Chem. Abst., 24: 1608, April 10, 1930. Analysis of corroded cast iron water pipes which had failed after 30 years service showed: iron, 44.5; carbon, 12.4; silicon, 9.4; phosphorus, 5.07; water (hygroscopic), 2.47; water (constitutive), 9.23; oxygen, 16.93 per cent. The author assumes that iron has not been lost, the low iron content being caused by increases in carbon, hydrogen, and oxygen. Humic acid derivatives of iron are believed to have been formed.—R. E. Thompson.

Welding in Chemical Engineering. J. R. BOOER. J. Soc. Chem. Ind., 49: 17–21T, 1930. From Chem. Abst., 24: 1607, April 10, 1930. Modern welding depends on the ability to produce and control temperatures ranging from 2000° to 3500°. Present day practices are explained from chemical and physical viewpoints.—R. E. Thompson.

Electric Welding. JAMES CALDWELL. Electrician, 104: 229, 1930. From Chem. Abst., 24: 1584, April 10, 1930. A brief review.—R. E. Thompson.

Corrosion of Metals and Alloys. ALBERT PORTEVIN. Rev. métal., 26: 606–31, 635–54, 1929. From Chem. Abst., 24: 1608, April 10, 1930. Review of the effects and modes of corrosion, the chief factors affecting corrosion, and the methods of estimating or characterizing corrosion. Following general conclusions are drawn: each corrosion problem must be considered on its own merits; chemical resistance cannot be reduced to a determination of other factors or properties, such as chemical composition; liability to corrosion must therefore be determined directly and its control cannot and must not be replaced by other requirements or conditions; in this direct determination, results of corrosion tests are comparable only provided that experimental methods and conditions are strictly identical and standardization is required in order that results of different investigators may be compared; it is necessary to guard against generalization of the results of individual corrosion tests, or of series of tests, and their application to a given type of alloy or process of manufacture.—R. E. Thompson.

Influence of Calcium on the Fauna of Peaty Waters. I. A. SMORODINTZEV and A. N. ADOVA. Bull. soc. chim. biol., 10: 1298–1305, 1928. From Chem. Abst., 24: 1671, April 10, 1930. Water from *Carex* swamps (average pH 7.728; average calcium content, 46.83 p.p.m.) contains large numbers of *Anopheles* larvae, while water from *Sphagnum* swamps (average pH 5.04; average calcium content, 10.61 p.p.m.) is invariably devoid of these larvae. Examination of other peaty waters shows that low calcium contents generally accompany low pH values and small numbers of larvae.—R. E. Thompson.

Regulations Regarding the Maximum Degree of Contamination Permissible in Sewer Waters Discharging into the Water Reservoirs. Purification of Waste Water from Tanning Industry. Trans. Central Comm. for Protection of Water Reservoirs from Pollution by Ind. Waste Waters (Moscow), No. 6, 165–6, 1927. From Chem. Abst., 24: 1691, April, 1930. The sewer water

should not (1) putrefy upon standing in a closed vessel, (2) give a strongly acid or alkaline reaction, (3) be strongly colored, or (4) contain more than 60 p.p.m. of suspended matter.—*R. E. Thompson.*

Barium Aluminate and Its Use in Water Purification. R. STUMPER. Chimie & Industrie, **22**: 1067–83, 1929. From Chem. Abst., **24**: 1687, April 10, 1930. The properties of barium aluminate and its suitability for boiler feed water purification were investigated. Freshly prepared 10 per cent barium aluminate is unstable; for several weeks there separates out a hydrated barium aluminate having a final composition of $BaO \cdot 10Al(OH)_3$, after which a species of equilibrium is attained. Reactions between the aluminate and impurities in the water readily occur, upsetting the equilibrium and resulting in highly complex precipitates, the complexity of which is increased by adsorption phenomena. Precipitation of sulfates by barium aluminate is practically quantitative, both in the cold and at 100°. Elimination of calcium is limited by the relatively high solubility and the hydrolysis of calcium aluminate. The solubility of calcium aluminate decreases with increase in temperature, favoring its deposition on heated surfaces. The reaction with calcium salts is considerably influenced by the anion present and calcium elimination is more complete, the less soluble the barium salt formed, i.e., the greater the amount of precipitate formed. Precipitation of magnesium by barium aluminate is more complete than that of calcium because of the lower solubility of magnesium aluminate and the formation of insoluble magnesium hydroxide. Magnesium aluminate has a positive solubility coefficient, so that it precipitates first. Elimination of magnesium is affected by anions in the same way as elimination of calcium. The complex precipitates of $BaSO_4 \cdot xCaO \cdot yAl_2O_3 \cdot 2H_2O$ carry down, either chemically or by adsorption, a certain amount of calcium sulfate; as much as 15 per cent of the calcium sulfate present may be thus entrained. The reactions with the principal incrustants are complex and cannot be expressed by simple equations. The action of barium aluminate on solutions of calcium and magnesium sulfates and bicarbonates is very important because of regenerescence phenomena due to successive reactions; these phenomena are all the more interesting since all natural waters fall into this class of solutions. Barium aluminate in presence of bicarbonates reacts as follows: $Ca(HCO_3)_2 + BaO \cdot Al_2O_3 + 2H_2O = BaCO_3 + CaCO_3 + 2Al(OH)_3$; following this a further reaction takes place: $BaCO_3 + CaSO_4 = CaCO_3 + BaSO_4$. The complex colloidal precipitates formed by barium aluminate entrain, by adsorption, all oil contained in water; it is necessary for this de-oiling action, however, that the water should have an alkaline reaction before addition of the aluminate, and that it be subsequently properly filtered. As the complex precipitates formed retain their adsorbing power toward certain impurities for some time, it is advisable to leave the sludge in contact with the water to be purified as long as possible and even to activate this contact by agitation or other means.— *R. E. Thompson.*

Operation and Control of Boiler-Feed Water Purification Systems. S. T. POWELL. Mech. Eng., 51: 935–40, 1929. From Chem. Abst., 24: 1688, April

10, 1930. Routine tests of value in the control of boiler feed water and of concentrated boiler waters are given. Various simply executed methods for determining the chemical nature of the feed waters are also given. Numerous tables show the cost of laboratory control and its relative value.—*R. E. Thompson.*

Some Results of Boiler Water Conditioning. R. E. HALL. Iron and Steel Eng., **6**: 633–4, 1929; cf. C. A., **23**: 3764, 4985, 5257. From Chem. Abst., **24**: 1688, April 10, 1930. Discussion and a reply.—*R. E. Thompson.*

Water Purification. J. W. BERRIMAN. Paper Maker & Brit. Paper Trade J., International No., 58–62, 1929. From Chem. Abst., **24**: 1687, April 10, 1930. A discussion and description of water purification methods and of control tests.—*R. E. Thompson.*

The Katadyn Method of Water Sterilization. CH. SCHWEIZER. Mitt. Lebensm. Hyg , **20**: 303–11, 1929; cf. C. A., **23**: 5524. From Chem. Abst., **24**: 1687, April 10, 1930. The Katadyn method kills pathogenic water organisms and *B. coli*, but does not seem effective for certain harmless bacteria, consequently absolute sterilization cannot be hoped for.—*R. E. Thompson.*

Water-Hammer Erosion. STANLEY S. COOK. Proc. Univ. Durham Phil. Soc., **8**: Pt. II, 88–100, 1929. From Chem. Abst., **24**: 1834, April 20, 1930. An attempt to show that in corrosive or erosive action promoted by the motion of metal surfaces, and frequently attributed to corrosion, a satisfactory explanation can be advanced by associating it with the phenomenon known as water-hammer. Rapid erosion of runners, casings, and liners of centrifugal pumps and hydraulic turbines can thus be accounted for. With steam turbines, since erosion is found only on moving blades it must be attributed to drops of water which are swept off the fixed blade into the path of the moving blades and do not partake of the general velocity of the steam. Pitting of brass condenser plates may be due to this cause.—*R. E. Thompson.*

Corrosion by Superheated Steam. J. K. RUMMEL. Iron Age, **124**: 1525–7, 1929. From Chem. Abst., **24**: 1834, April 20, 1930. Rates of corrosion of various steels in superheated steam were determined for steam pressures varying from 220 to 3450 pounds per square inch. Corrosion losses within the tubes were measured by determining hydrogen evolved. Gas sampling and analyzing methods were developed. Data from tests on various commercial boiler installations indicated that in oxygen-free steam corrosion of steel tubes in superheaters is negligible for steam temperatures up to 800°F. The iron-water reaction does not limit the use of KA 2 up to a temperature of 1200°F. on the fire side. Hydrogen evolution and corrosion in general are accelerated by presence of oxygen in the feed water. Within limits, alkalinity does not stop corrosion due to hydrogen evolution, or corrosion due to oxygen.—*R. E. Thompson.*

Pipe Line Corrosion. K. H. LOGAN, WALTER ROGERS and J. F. PUTNAM. Gas Age-Record, **64**: 899–903, 907, 1929. From Chem. Abst., **24**: 1835, April

20, 1930. The authors found that pipe-line corrosion occurs in a number of localities where there is no stray current electrolysis. A study revealed the fact that lines carried small currents, which they designate as pipe-line currents, and that in most cases corrosion was severe where these currents left the line.—*R. E. Thompson.*

The Estimation of Phenol and Cresol by Bromine. H. Ditz. Z. Anal. Chem., 77: 186; Chem. Zbl., 1929, 2: 5, 613. Critical account of published work of Järvinen. Instead of drying phenol mixture as recommended by Järvinen, fractional distillation of phenol remaining after ether is distilled off is suggested. Remaining traces of ether and water can be removed by raising temperature twice or thrice to 180°.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

The Detection of Aluminium by Color Reactions. R. Eegriwe. Z. Anal. Chem., 76: 438; Chem. Zbl., 1929, 2: 5, 609. Color reactions employed for detection of aluminium are orange-red color with yellow solution of alizarin S., and greenish fluorescence with pale yellow alcoholic solution of morin. Author alters second method, by adding saturated solution of morin in methyl alcohol (0.1 to 0.2 cc.) at room temperature to 1 or 2 drops of aluminium chloride solution and then adding sodium acetate to supersaturation. Two new reactions of aluminium with alizarin red PS and eriochromcyanin R. are described. Detection of aluminium by morin, alizarin S, and alizarin red PS is easily influenced by iron. Eriochromcyanin R. is most satisfactory reagent in presence of iron.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

The Heat Conductivity of Boiler Scale. C. Eberle and C. Holzhauer. Arch. Wärmewirtsch., 9: 171; Chem. Zbl., 1928, 2: 13, 1248. Tests were made with synthetic boiler scales to determine change in heat conductivity with alterations in temperature, density, and composition. Temperature changes had little effect, but density, much; composition affected density. At lower densities different kinds of scale (gypsum, carbonate, silicate) showed little difference; but at higher densities silica compounds were better conductors than gypsum which was slightly better than lime. As gypsum scale is always formed in state of high density, it is the least dangerous for boiler work. Silicate scales form at low densities and then have little conductivity and are most dangerous. Structure and crystallization of different scales are discussed.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

Base-exchanging Substances; Adsorbents. F. B. Dehn. E. P. 313, 206 Ill. Off. J. Patents., 1929, No. 2115, 3929. Base-exchanging substances may be produced by reaction of sodium aluminate with sodium silicate with addition of acid, which can be added to reaction mixture or to final product any time before complete drying. Sufficient acid should be added to neutralise most or all of free alkali present, and reaction mixture should be neutral to phenol-

phthalein and alkaline to methyl orange. The solution employed should have consistency of gel. Porosity of product renders it useful also as an adsorbent. —*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

Base-Exchanging Substances. Reymersholms Gamla Industrie Aktiebolag, E. P. 313, 522. Ill. Off. J. Patents, 1929. No. 2116. Hydrated base-exchanging substances can be prepared by boiling under pressure the residue of "waste clay," consisting of silicic acid and iron and aluminium oxides with oxides, hydroxides, or carbonates of the alkali or alkaline earth metals. "Waste clay" is obtained when natural silicates, such as kaolin, bauxite, or refractory clays, are decomposed with acid. If lime is used as alkaline agent, hard products are obtained. Prior to use, product may be washed with solution of common salt until free from lime.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

Factors Influencing the Toughness of Coagulated Matter. JOHN R. BAYLIS. Water Works and Sewerage, **77**: 184–8, 1930. Various factors influence strength of coagulated matter. Micro-organisms have greatest influence in shortening filter runs. Gelatinous precipitates of organic origin influence strength of coagulated matter. Increase of temperature increases the strength; but it is not desirable to increase temperature of the water. Changes in chemical treatment may produce stronger flocculation. Flocculated turbidity must be very low in water going to filter beds when temperature is low, to avoid flocculated matter passing the beds, unless there is something in the water that imparts toughness to flocculated matter.—*C. C. Ruchhoft (Courtesy Chem. Abst.).*

The London, Eng., Metropolitan Water Board. Anon. Water and Water Engineering, **32**: 384, 552, December 20, 1930. Some figures in twenty-seventh annual report are of general interest as indicating immensity of problem of supplying London with water. One hundred seventy-eight slow sand filter beds, covering almost 175 acres, are required to filter the water; in addition, there are 51 primary filters. Ninety-three service reservoirs, only two of which are uncovered, with capacity of 322 million gallons, serve as storage basins. Two hundred seventy-five engines, with total of almost 49,000 horse-power, do the pumping. Population supplied is about 7.5 millions and rate of consumption, 36.4 imp. gallons (43.8 U. S. gallons) per person per day.— *Arthur P. Miller.*

Accrington, Eng., Water Supply Works. Inauguration of New Works at Altham and Burnley Road by Rt. Hon. Philip Snowden, M.P. Anon. Water and Water Engineering, **32**: 384, 553–554, December 20, 1930. New works include new borehole to water-bearing strata about 300 feet from surface, pumping plant, reservoir of one-half million gallons capacity, and mechanical filtration plant. Borehole is lined with thick steel tube, exteriorly cement-grouted to exclude extraneous water. Pumping plant is of usual borehole

type. In construction of reinforced concrete reservoir, granite chippings were used with sand and cement—*Arthur P. Miller*.

The Reliability of Rainfall over the British Isles. JOHN GLASSPOOLE. Water and Water Engineering, 32: 384, 555–562, December 20, 1930. Writer discusses at some length variability of rainfall in British Isles, basing his statements on information obtained during years 1870–1927 at forty recording stations. He concludes that in England and Wales wider variation between extremes of rainfall is to be expected than in either Scotland, or Ireland. Extremes in rainfall have not occurred simultaneously over the country and, consequently, extremes over a large area are likely to be less severe than those of individual stations. Driest periods occur in spring and summer and wettest, in winter and autumn.—*Arthur P. Miller*.

The Kempton Park Primary Filters of the London, Eng., Metropolitan Water Board. S. WALKER. Water and Water Engineering, 32: 384, 562–567, December 20, 1930. Gives information regarding design and construction of new filters, including details concerning concrete mixture used and other similar points. One interesting feature is pebble finish given to external walls of filters, on the assumption that surface discolorations due to any slight leakage through the walls would tend to show less conspicuously on this, than on a smooth finish. Doubt exists as to whether this will actually be the case. Cost of works and new pump installation are also discussed.—*Arthur P. Miller*.

The Hollerith System for the Analysis and Tabulation of Expenditure. G. O. RITCHIE. Water and Water Engineering, 32: 384, 567–571, December 20, 1930. By this electrical tabulating and accounting system, the maximum use of figures can be obtained quickly, accurately, and economically. Application to a large water works department is described. System divides itself into three mechanical processes; punching, sorting, and tabulating. By punching process, record of each item takes the form of standard card, suitably punched. Sorting separates the cards, irrespective of their original order, into predetermined groups, totals of which are desired. Tabulating process obtains these totals. Apparatus used is pictured. System has been successfully applied by London, Eng., Metropolitan Water Board to tabulating wages and respective costs of pumping, filtration, meter repair, and transportation.—*Arthur P. Miller*.

Corrosion in Underground Iron Structures. MICHAEL J. BLEW. Public Works, 61: 9, 31, 1930. The variables affecting corrosion of metal underground include current density, moisture content of soil, presence of dissolved gases, mechanical and chemical properties of soil, temperature, polarization, formation of high resistance films, changes in polarity, and changes in frequency. Corrosion may be reduced by selection of resistant materials, use of protective coatings, and by limiting voltage drop in negative return lines.—*C. C. Ruchhoft (Courtesy Chem. Abst.)*.

Cross-Connections in Chicago. Anon. Public Works, 61: 118, 1930. During 1928, 874 Chicago premises were inspected: cross-connections between

city mains on the one hand and on the other, the river, foul wells, cooling towers, elevator pits, lime vats, salt brine vats, condensers, etc. were disclosed; also, city water pipes were found submerged in foul water vats, sterilizers, dish washers, and other containers of polluted water. Leak in condenser tube of an ammonia cooler caused complaint of blue water. Analysis showed presence of 1.76 grams of ammonia and 0.0035 grams of copper per 100 cc.—*C. C. Ruchhoft* (*Courtesy Chem. Abst.*).

Portable Laboratory Kit for Filter Plant Control. K. W. GRIMLEY. Public Works, **61**: 141–42, 1930. Wooden box, 10 by 10 by 21 inches, holds all necessary equipment for making following chemical tests: (1) alkalinity; total, carbonate, bicarbonate, and caustic; (2) approximate hardness; total, carbonate, and non-carbonate; (3) pH; (4) free CO_2; (5) residual chlorine; and (6) residual aluminum sulfate. Complete kit weighs 25 pounds.—*C. C. Ruchhoft* (*Courtesy Chem. Abst.*).

Water Supply of Northern New Jersey. Anon. Public Works, **61**: 162–4. 1930. Discussion of features of North Jersey District Water Supply Commission, recently created to provide adequate water supply for eight municipalities. Plan includes division of metropolitan district into zones according to elevation and establishment of principle of joint use and joint ownership of water supplies. All existing and new supplies will be interconnected and will be operated as unit through some co-ordinating agency. Operation is to be at cost, with due allowance made for elevation and location of various consumers.—*C. C. Ruchhoft.*

Detroit's Springwell Pumping Station. Anon. Public Works, **61**: 34, 1930. Water will flow from Detroit river through 14-foot tunnel to well 56 by 100 feet, from which it will be pumped to filters. Filter effluent will flow to high-lift pumps which will deliver it directly to mains.—*C. C. Ruchhoft.*

Pre-Ammoniation at Springfield, Illinois. CHAS. H. SPAULDING. Public Works, **61**: 35–6, 1930. Use of ammonia since 1929 for prevention of chlorotastes has led to following inferences: pre-ammoniation increases chlorine efficiency; prevents chlorinous and chloro-phenol tastes; is simply and cheaply applied; causes no undesirable after effects; and permits chlorine residuals sufficiently high to overcome subsequent contamination. With high pH values, as much as eight hours time of contact is necessary, if maximum sterilization is required.—*C. C. Ruchhoft* (*Courtesy Chem. Abst.*).

Building Reservoirs Over Coal Mines. Anon. Public Works, **61**: 29–30, 1930. The only site available for Pittsburg reservoirs was found to be underlaid with old coal workings. Experiments showed feasibility of grouting the cavities with concrete sufficiently to stiffen the material adjacent to each drill hole. The 193 churn-drilled holes were laid out so as to concentrate somewhat under walls of reservoir. Grout was placed with pneumatic grout machines equipped with agitator paddles. Most of grout used had proportion of one bag of cement to 3 cubic feet of sand. In all, 91,711 cubic feet of grout was

placed. Concrete reservoirs were adopted in order to utilize, by their shape, all of available area. Details of construction are given, with special reference to water-tight construction joints.—*C. C. Ruchhoft.*

The Determination of Total Alumina in Filtered Water. F. O. BALDWIN. Water Works and Sewerage, **77**: 311, 1930. New method for determination of unprecipitated alum is simple, rapid, and sensitive to one part in ten million as Al_2O_3. *Reagents* required: (1) Three-tenths per cent aqueous solution of alizarin red S. (alizarin sodium monosulphonate) in 2N sulphuric acid; (2) Saturated solution of sodium bicarbonate. Filter through alumina-free filter paper and add 2 cc. distilled water per 100 cc. of filtrate. Strength of this soda solution should be held close to normal, that is, 8.4 per cent $NaHCO_3$ (Arm & Hammer bicarbonate may be used); (3) Fifty per cent acetic acid, made with highest grade reagent and boiled distilled water. *Procedure.* To 50 cc. of freshly-collected and well-shaken water add 1 cc. of No. 1. Use phosphoric acid flasks, size 200 cc. Boil 2 minutes on hot plates; neutralize with No. 2 to just barely purple, adding same very carefully, drop by drop; then add exactly 0.5 cc. excess, boil 2 minutes, and cool in cold water. When cold, add 1 cc. of No. 3, let stand 1 minute, and compare with color standards sealed in similar 200 cc. Pyrex flasks. Standards must be prepared very accurately and sealed under partial vacuum.—*C. C. Ruchhoft (Courtesy Chem. Abst.).*

Water Supply Problems in Holland. F. A. LIEFRINCK. Public Works, **61**: 9, 19–20, 1930. Water has been one of the enemies of the inhabitants of the Netherlands. Much of it is salt or brackish. Due to unfavorable geo-hydrological conditions, difficulties are encountered in providing potable water. Sixty-one per cent of the population, divided over 444 parishes, now have piped supplies. Rural supplies are built by combinations of several parishes. Government Water Supply Bureau coöperates, even assuming a certain share of financial risk in the case of new and fully approved undertakings. Of 444 supplies, 142 are dune ground water, 253, other ground water, and 49, river water. The dune ground waters are typical lens-shaped fresh water accumulations. The water is captured either by means of shallow wells, as at Haarlem, of in open channels, as at Amsterdam, or in covered drains, as at the Hague. Future problems are now arising, due to reclamation of the Zuider Zee which will add some 550,000 acres of fertile land to the country. One of the four polders will be ready for occupation in 1931. Soil of these new polders has been under salt water for centuries so that there is little hope of finding there fresh water for the population who will come to settle.—*C. C. Ruchhoft (Courtesy Chem. Abst.).*

Building the Hydraulic Fill Saluda Dam. ARTHUR R WELLWOOD. Public Works, **61**: 9, 26–7, 1930. Saluda dam, for hydro-electric power development, is of semi-hydraulic fill type, consisting of two outer dykes built of dry earth dumped from wooden trestles, between which a bore is formed by sluicing earth from inside faces of dykes. Dam has maximum height of 212 feet, crest length of 7838 feet, crest width of 25 feet, and maximum width at base of 1150 feet and contains 11,000,000 cubic yards of earth.—*C. C. Ruchhoft.*

Chemical Handling at Dalecarlia Water Filtration Plant, Washington, D. C.
Anon. Water Works and Sewerage, 77: 316, 1930. Storage of chemicals is in
a square building vertically divided into nine squares, thus providing eight
bins around the center space, which is used for elevator shaft and stairway.
In tower over bins is housed pneumatic receiving station, from which
incoming material can be discharged to any desired bin through gravity chute.
Detailed description of system is given.—C. C. Ruchhoft (Courtesy Chem.
Abst.).

The Taste Problem Solved. JOHN R. BAYLIS. Water Works and Sewerage,
77: 299–304, 1930. Elimination of tastes in water has been along two lines.
One is to prevent the taste from becoming more pronounced when chlorine is
added to the water and the other is to remove from the water all bodies which
cause objectionable tastes by changing them to inoffensive compounds. The
success of superchlorination treatment depends upon (1) the power of chlorine
to break down offensive compounds and (2) effectiveness of dechlorinating
process. The most common tastes, such as those produced by microörgan-
isms and by phenolic bodies, are readily changed to inoffensive compounds
with chlorine. With superchlorination all bacterial life is destroyed, oxidiza-
ble organic matter is reduced, and the water is more likely to remain stable
when stored. Sulphur dioxide is very effective dechlorinating agent. In
practice, it is best to add same amount of sulphur dioxide as is present of
residual chlorine. Sodium bisulphite is also used for dechlorination. The
most successful treatment for removal of tastes is the chlorine activated-
carbon process. The water is superchlorinated and a residual of 0.5 to 1.0
p.p.m., prior to dechlorination, is sufficient if there has been 3 to 4 hours'
contact time. Charcoal, lignite, and activated carbon are dechlorinating
agents. Activated carbon is best suited for the purpose and removes not only
excess chlorine, but also offensive compounds not changed by the chlorine.
Another process is the ammonia-chlorine treatment. Ammonia is added to
the water prior to chlorine. This is done to prevent chloro-tastes. Chlorine
unites with ammonia to form chloramine, a weak oxidizing agent which does
not readily oxidize organic material present in water. Chloramine is a more
effective bactericide than chlorine; it prevents after growths; it eliminates
objectionable odors and tastes; it allows 50 per cent reduction of amount of
chlorine required for actual sterilization; it retards oxidizing action of chlorine
and prevents its absorption, thus allowing practically all the chlorine to be
available for germicidal action. More time is required to sterilize with chlora-
mine than with chlorine. Potassium permanganate has been used to eliminate
taste-producing compounds, but it gives the water a slight bitter and astrin-
gent taste. Ozone has not been used extensively to eliminate tastes, due to
its cost, but it may be quite effective.—C. C. Ruchhoft (Courtesy Chem. Abst.)

Water Treatment Works of Iola, Kansas. M. P. HATCHER. Water Works
and Sewerage, 77: 285, 1930. Present supply, from Neosho River, is settled
and chlorinated. Hardness varies from 100 to 500 parts per million, 65 to 75
per cent of which is due to calcium carbonate, while magnesium is usually
lower than 20 parts per million. During low flow, chloride content is as high

as 500 to 700 parts per million, due to oil-field pollution; but for most of the year chloride content is less than 400 parts per million. New filtration and water-softening plant is designed to treat 2 million gallons per day, with provision for further extension. Raw water will be delivered by present low-service pumps to mixing chamber below chemical application room. Lime is applied at entrance to this chamber where water has retention period of about nine minutes and velocity of 0.8 foot per second. Water next goes to reaction tanks for about 30 minutes and thence to existing preliminary settling basins for three-day retention period. It is then aërated and afterwards carbonated in special chamber. Aëration is by means of Sacramento spray nozzles, and carbon dioxide from natural-gas generating plant in chemical head house is distributed through grid system on floor of carbonation chamber, where 30-minute retention period is provided for. Water is then treated with alum and goes to filters, of which there are three, each capable of filtering 667,000 gallons per day. Storage is provided for 215,000 gallons. Chlorine is applied at entrance to last reservoir.—*C. C. Ruchhoft (Courtesy Chem. Abst.).*

The Hetch Hetchy Water and Power Project. M. M. O'SHAUGHNESSY. Water Works and Sewerage, 77: 295–98, 1930. San Francisco's mountain water and power project is well under way. Storage reservoirs at Hetch Hetchy and Lake Eleanor, with Sierra Nevada tunnels and Moccasin power house, are producing $2,000,000 worth of hydro-electric energy annually. Foothill tunnel, 15.8 miles long, was completed in 1929. Dam across Moccasin Creek is 850 feet long and 51 feet high, contains 143,000 cubic yards of fill and 3600 cubic yards of concrete and has impounding capacity of 648 acre-feet. At Red Mountain Bar, aqueduct is of steel pipe 9 feet 6 inches in diameter, lined inside with cement and covered with envelope of concrete about 2 feet thick. At this point a $1,000,000 power house is contemplated, which will produce power valued at $500,000 annually. From end of Foothill tunnel, 47-mile pipe line will be built across San Joaquin valley. Aqueduct pierces Coast Range as a tunnel 28 miles long. The present tunnel will have capacity of 300 million gallons per day. Excavation is by drilling and blasting. Lining of tunnel is placed by concrete gun. Water-bearing seams are encountered and occasional methane and hydrogen sulphide gas pockets must be guarded against. System when completed in 1932 with capacity of 300 m.g.d. will have cost about $80,000,000. City has recently acquired for $40,000,000 from Spring Valley Water Co. 62,000 acres of watershed and reservoir lands, five impounding reservoirs with 65 billion gallons capacity, well system and infiltration gallery, 111 miles of main aqueduct, distribution system of 20 reservoirs and 750 miles of pipe line, and over 105,000 consumers: this system has estimated yield of 65 million gallons per day.—*C. C. Ruchhoft.*

Albany's New Water Supply. Anon. Public Works, 61: 41, 1930. Present supply of filtered Hudson River water for Albany will soon be replaced by one drawn from impounding reservoirs located twenty miles distant. The six-million-dollar project includes two impounding reservoirs and dams, twenty miles of 48-inch cast iron pipe line, filter plant, and 100-million-gallon distribution reservoir. Construction details of dams and of pipe laying are included—*C. C. Ruchhoft.*

Recent Progress in Water Supply. Report by C. A. Emerson, Jr., H. B. Cleveland, H. F. Ferguson, J. C. Pritchard and Robert Spurr Weston. Public Works, **61**: 23–4, 1930. Discusses advances made in methods for improvement of water color, taste, and odor and of softening. For color removal, superchlorination followed by dechlorination is effective when iron content of coloring matter is high. Chlorinated copperas has been used effectively, as also have filters loaded with ferric hydrate. Hydrogen-ion control and improved mixing devices have decreased cost of color removal. Progress has been made in eliminating mud balls from filters. Use of air during filter washing has been recently re-introduced. Filter sand may be kept clean by employing velocities sufficient to produce 50 to 60 per cent expansion of sand bed. Variations in temperature must be corrected for by varying wash water velocity. Cleaning of filter sand may be accomplished by use of caustic soda. Improved methods for recovering phenols at coking plants have reduced considerably tastes and odors caused by industrial wastes. Activated carbon filters are especially effective for removing objectionable odors and tastes. Another method is pre-ammoniation. Improvements in filter plants include mechanical devices to replace cleaning by hand and utilization of coagulating effect of sludge from coagulating basin on silt contained in raw water. Zeolite and lime-soda water-softening processes have been improved by adoption of reaction tanks, settling basins, and mechanical mixers and conveyers.— *C. C. Ruchhoft (Courtesy Chem. Abst.).*

Manchester, England, Water Supply Pipes. Anon. Water and Water Engineering, **33**: 387, 101, March 20, 1931. Pipes made of new ternary lead alloy discovered by British Non-Ferrous Metals Research Association are used. Composition is 98.25 per cent lead, 1.5 per cent tin, 0.25 per cent cadmium, and pipes are two thirds the weight of lead pipe, have tensile strength 84 per cent greater, resistance to vibration 217 per cent greater, and exhibit greater resistance to corrosive waters. They save 22 per cent in cost of metal, as compared with lead besides the saving on transport, since, weight for weight, the alloy gives 33.3 per cent greater length than lead, with superior mechanical strength.—*W. G. Carey.*

Skipton, England, Waterworks. Anon. Water and Water Engineering, **33**: 387, 103–105; March 20, 1931. Moorland water is impounded in 175-million (Imp.) gallon reservoir and is soft, acid with peat acids, has distinct stain and seasonal turbidity. Candy filters are installed and dosage of alum and of lime is regulated by air contained in cylinder and automatically compressed to equivalent of upstream pressure by connection to upstream side of Venturi tube in main. Alum solution is added on one side of Venturi throat and lime on the other. Colour is reduced from 80° to 6.5° Hazen, and pH value increased from 6.5 to 8.5.—*W. G. Carey.*

Substances Producing Taste in Chlorinated Water. B. A. Adams. Water and Water Engineering, **33**: 387, 109–113; March 20, 1931. Iodoform taste is due to lower monohydric phenols of benzene series, their mono-carboxylic, aldehydic, alcoholic derivatives, or their inorganic salts. Tastes more nearly

resembling those met with in actual practice are produced by phenol, mono-hydric phenols with substituted ortho-groups, or inorganic iodides. Para-compounds yield tastes less resembling iodoform, but of greater pungency than meta-compounds. Strongest tastes are with phenol, o-cresol, salicylalde-hyde, and o-hydroxybenzyl alcohol. The tastable products are probably mixtures of o-chlorophenol, p-chlorophenol, trichlorophenol, or homologues, o-chlorophenol having strongest taste and resembling iodoform.—*W. G. Carey.*

Hydro-electric Power in Sweden. E. UPMARK. Water and Water Engi-neering, 33: 387, 123–128; March 20, 1931; and 388, 177–180; April 20, 1931. Items from a comprehensive survey show that Sweden ranks third in water power resources in Europe, with 6.5 million kw. There are 15 principal supply areas containing 23 hydro-electric stations having capacities over 10,000 kw. Details of several of larger power plants are given.—*W. G. Carey.*

Water Tower and Reservoir, Southall, England. Anon. Water and Water Engineering, 33: 388, 160–164; April 20, 1931. Reservoir, holding a million (Imp.) gallons, is circular, of reinforced concrete, with internal diameter of 103 feet and depth of 20 feet, 10 feet of which is below ground. Circular water tower forms integral part of reservoir, rising therefrom to height of 56 feet, and has capacity of 400,000 (Imp.) gallons. Central axis shaft for tower forms in-termediate support for tank, is 25 feet in diameter, and contains the pipes. Such an arrangement of water tower centrally above the reservoir is economi-cal and appearance is good. Details are given of the construction, together with sectional elevation and photographs of completed structure.—*W. G. Carey.*

Halifax (England) Corporation Filtration Plant. Anon. Water and Water Engineering, 33: 388, 153–159; April 20, 1931. One of largest installations of pressure filters in operation in Europe; is designed to deal with 12 million (Imp.) g.p.d. and consists of 84 Paterson pressure filter units, each 9 feet in diameter, with total filtering surface of 5340 square feet. The water, from 10 impounding reservoirs (further two being under construction), is acid and contains peaty matter in suspension; it is treated with aluminium sulphate and lime, automatically proportioned to flow of water. Flow is measured by Verturi meter which actuates mercurial differential gear operating multi-stage rheostat, which in turn regulates motor driving the chemical plunger pumps. Visible flow meters, pressure gauges, and clarity indicators, finished with chromium plating, are mounted on central control stand. Operation and cleansing of filters with compressed air and with water are described in detail and illustrations of plant are given.—*W. G. Carey.*

Burst Water Mains. H. J. F. G. Water and Water Engineering, 33: 388, 151–2; April 20, 1931. Report of London, Eng., Metropolitan Water Board states that bursts in cast-iron mains may be due to (1) removal of lateral or vertical support by other users of subsoil, (2) variation in nature of support, (3) direct blows from traffic, transmitted through walls of valve pit, or through

solid concrete over the main, (4) subsidence caused by deep foundations for adjoining buildings, or pumping operations connected therewith, (5) vertical contact between mains, (6) water hammer, (7) shock, and (8) faulty castings. Defects due to any of these may not show immediate leakage, but rapid change of temperature of water may complete the damage. M. W. B. practice is 3 feet cover for pipes up to 21 inches diameter; 3½ feet, from 21 to 33 inches; and 4 feet for larger sizes. Yellow London clay has been responsible for bursts and special precautions, e.g. concrete casing, are taken where necessary. In all future arterial roads, or considerable widening or reconstruction of traffic arteries, subways should be provided. Steel pipes adequately protected should be used for trunk and large high pressure pumping mains, but jute hessian dipped in bitumen is not adequate protection.—*W. G. Carey.*

Researches into the Plumbo-solvency of Water Fittings. Anon. Water and Water Engineering, **33**: 388, 149; April 20, 1931. In investigation at Leipzig, Germany, it was found that amount of lead dissolved increased with extent of metal surface exposed and with duration of contact. Dissolution was greatest with river water, which always contains acids and organic matter; carbon dioxide also increases lead solubility. Trouble with Leipzig water, which is free from organic matter, is due to its high percentage of carbon dioxide and only remedy is to treat it with suitable reagents.—*W. G. Carey.*

The Direct Count of Colon-Aërogenes Organisms, Its Possibilities and Promise. FRED O. TONNEY and RALPH E. NOBLE. Jour. Inf. Dis., **48**: 4, 413–417, April, 1931. Authors summarize some of practical advantages of direct plating method in cyanide-citrate agar. (1) Greater simplicity than fermentation methods: it consists only of planting the plates and counting them after incubation. (2) It yields results in shorter time: completed observations are obtained in from forty-two to forty-eight hours: presumptive indications are available in thirty-six hours. (3) It gives additional information, namely, separate *Bact. coli* and *Bact. aërogenes* counts and their numerical relation to each other, as well as total count. (4) Plating gives more accurate index than is obtainable by use of liquid mediums: direct counts are not obscured by overgrowths and are not interfered with by other common lactose-fermenting organisms, such as *Clostridium welchii*, or *Bacillus aërosporus*. (5) The "pour plate" process is an excellent method of isolating pure cultures of *Bact. coli* and *Bact. aërogenes*: deep colonies fished from poured plate are more apt to yield pure strains than are surface colonies picked from Endo's medium, or eosin-methylene blue agar, inoculated by surface streak method.—*R. E. Noble.*

Sanitary Control of Swimming Pools as Practiced in Wilmington, Delaware. RUSSELL T. DES JARDINS, Municipal Sanitation, **2**: 5, 221–222, May, 1931. Sanitary control of public swimming places is vested in public health authorities, to prevent spread of disease and to protect the individual bathing place from a reputation for unhealthfulness arising from false statements. Following standards are in force. (1) Bacterial count of less than 100 per cc. (2) Not over 20 *B. coli* per 100 cc. (3) Residual chlorine of from 0.3 to 0.5 p.p.m. (4) Proper control of alkalinity, with pH range of from 7.4 to 8.0. (5) Either

constant recirculation, or proper emptying and cleaning of pools at frequent intervals. (6) Sanitary control of all locker rooms, toilets, and other facilities, with occasional sterilization of locker room floors and runways. Facsimile swimming pool score cards are included.—*R. E. Noble.*

NEW BOOKS

The Construction of Wells and Boreholes for Water Supply. By J. E. DUMBLETON (1928). Published by Crosby Lockwood and Sons, London. 134 pages. Price 6 shillings. The author states that this book was intended to be a revised edition of the Rudimentary Treatise on Wells and Well-Sinking by Swindell and Burnell, but since the alterations were considerable, a new book was written. The book is written in a style which makes it easy and interesting to read. It is more a history of well drilling than a description of present day methods. Subjects related to wells and well drilling are discussed briefly, and include the theory of springs, the various types of pumps for use in wells, the properties of water, and water analysis and purification. The book is suited for someone who desires to obtain an elementary knowledge of wells, the construction of wells, and water analysis and treatment. The first part of the book gives a few examples of the antiquity of wells and mentions one well which was constructed about 4,000 years ago. The chapter on the yield of wells gives suggestions for locating well. The statement is made that abundant proof is available to show that water divining is a reliable means of detecting the presence of flowing water near ground level. Most of the material on the construction of wells is concerned with digging wells by hand and drilling wells with pole tools, both of which methods are not used very often in the middle West, at the present time. However, since pole tools are not in common use the illustrations are of interest. In the discussion of deep-well pumps, mention is made of the construction of a central shaft, containing a centrifugal pump driven from the surface, and tunnels leading from the shaft to the wells such as were built at Rockford, Elgin, and Argo, Illinois.—*H. L. White.*

Proceedings of the Twelfth Texas Water Works School, 1930. The Water Supply of Abilene, Texas. WYNKOOP KIERSTED. Pp. 5–15. Discussion of proposed plan to develop ground water supply in valley of Elm Creek from formation fed mainly by leakage from an impounded supply, together with comparison of merits and cost of this project as compared with impounded supply. **Improvements in Waterworks and Sanitary Sewer Systems of the City of Dallas.** JNO. M. FOUTS. Pp. 15–21. Historical review of developments of sewerage and water supply systems of Dallas with description of plans for needed improvements. **Sanitation and Improvement Program of the Houston Water Supply.** J. A. SAULS and C. R. HARVILL. Pp. 21–25. Houston's supply is obtained from 48 wells ranging from 200 to 1500 feet in depth. Considered safe until appearance of contamination following flood in Buffalo Bayou showed need of sanitary control. Program includes: chlorination of wells and reservoirs to disinfect; continuous chlorination of supply, with apparatus in triplicate at Central Plant and in duplicate at others; syste-

matic periodical flushing of distribution system; chlorination of new mains; elimination of cross-connections; protection of wells and reservoirs; and close laboratory control. **Discussion on Chlorination in the Sanitary Program of the City of Houston.** G. A. Phillips. Pp. 25–27. Flooding of wells and reservoirs necessitated emergency measures for sterilizing supply. A 15,000,000-gallon reservoir was sterilized by adding chlorine direct from cylinders placed at manholes. Private supplies were disconnected from public system. Chlorinators were installed at the eight pumping stations, some equipment being borrowed from city high school swimming pools until regular equipment could be obtained. Experience since shows value of duplicate installations. **Water Supply Conditions at Sweetwater, Texas.** H. N. Roberts. Pp. 27–31. Water shortages in 1927 and 1928 showed urgent need of increased supply. Development of ground water supplies tided over emergency period. Impounded supply with storage capacity of 4 billion gallons is to be developed. **The Cross-Connection Control Program in Texas with Typical Illustrations of the Work in Dallas.** W. N. Dashiell, with discussion by H. E. Halpin. Pp. 31–46. In 1929 Dallas with coöperation undertook a detailed study of cross-connections between public supply and other systems with view toward their elimination. Methods and results of survey are described in detail. Discussion describes various methods of protecting such connections and reviews requirements and activities of various State Boards of Health on this matter. **Lake Watershed Protection and Sanitary Control of Well Sources.** X. H. Goodnough. Pp. 46–52. Contaminated surface supplies have frequently occasioned serious epidemics. Development of outdoor recreation promoted by the automobile has increased possibilities of contamination of impounded supplies and the demand that such supplies be made available for recreational use. Extent of control to be exercised and area of land which it may be desirable for water company to acquire will necessarily vary with circumstances, and in each case policy to be adopted must be selected in light of prevailing conditions. Ground waters are usually of better quality as regards safety; but geological conditions vary and should be investigated for each supply. In all cases frequent analyses should be made to check quality. **Development in the Treatment of Highly Polluted Water and in the Elimination of Bad Tastes and Odors.** John R. Baylis. Pp. 52–61. Tendency seems to be toward using filters to clarify water and to depend on chlorine to remove remaining bacteria. Treasury Department Standard was made more rigid in 1925. Studies made by U. S. Public Health Service have shown limits of pollution as indicated by *B. coli* content which can be removed by various methods of treatment. Much is being done to reduce pollution of streams, but there are some localities where it does not seem possible to reduce pollution to point where ordinary treatment process will handle the water without having to add chlorine to the extent of producing disagreeable tastes. Real progress is being made in dealing with such highly polluted waters. Superchlorination followed by dechlorination is useful in some instances. Activated carbon, ammonia, and permanganate are being used for removal of tastes and odors. **Swimming Pools and Waterworks.** Arthur M. Crane. Pp. 61–63. Brief comments regarding pools and regulations. **Joint State and Federal Underground Water Survey.** Walter N. White. Pp. 63–78. Ground waters con-

stitute important source of supply; knowledge of their volume and character is essential. U. S. Geological Survey coöperates with State agencies in investigating ground water resources. Survey is being conducted at present in Texas. Outline is given of available methods of estimating ground water supplies. **Water Supply Pumping.** HENRY H. BATJER. Pp. 78–84. Discussion of principles involved in selection of kind and size of pumps for water supply installations. **Key Rate Reduction.** F. J. FERRIER. Pp. 84–85. Outline of basis of key rate (fire insurance) of city or town. **Recent Developments in Preserving Standpipes and Other Metal Tanks.** D. W. PYLE. Pp. 85–87. Importance of painting steel for protection is stressed. Necessary to use electro-motor-driven steel wire brushes and caulking hammers for cleaning and air spray for painting, to reach crevices and joints. Sanitary flush valve should be installed in hemispherical bottom of water tank or tower. **Home Made Alum at Kansas City, Kansas, Filter Plant.** L. B. MANGUN. Pp. 87–93. This plant manufactures its alum by so-called HOOVER process. Detailed description of plant and of methods of manufacture is given, with cost data and comparison with commercial filter alum. **Chlorination Practices and Policies.** ARTHUR E. GORMAN. Pp. 94–98. Chlorination of water supplies is now general practice, but should not be sole dependence for polluted supplies, except in emergencies. Chlorination equipment should be in duplicate and dosage should be checked by *ortho*-tolidine test. Automatic equipment should be used where rate of flow is variable. Uses of chlorine for purposes other than sterilization are discussed. **The Open Reservoir and Bacterial Aftergrowths.** GEO. F. GILKISON. Pp. 98–101. Supply of Kansas City, Mo., after treatment comprising presedimentation, coagulation, filtration, and chlorination, is pumped to distributing reservoirs, several of which are uncovered. Treatment plant effluent shows no *B. coli*, but effluents from these storage basins show *B. coli* almost uniformly. It has been necessary to cut these basins off the line, as funds are not available for covering them. **The Water Works of the City of Mexico.** EDUARDO MOLINO, ANTONIO CORNEJO and ALFONSO V. ACOSTA. Pp. 101–117. Supply is obtained from wells and collecting galleries about 20 miles from city, and is chlorinated. Brief historical review of city's development and detailed description of the various parts of water supply system. **Simple Method of Testing for Residual Chlorine.** JANE H. RIDER. Pp. 118–19. White enameled cups are painted on inside to simulate colors produced by *ortho*-tolidine with 0.1 ppm. and 0.2 ppm., respectively, of residual chlorine. A third plain white enameled cup is used for sample to be tested. Details are given for preparing standards, collecting samples, and making tests. **Flood Relief Work in New Mexico.** PAUL S. FOX. Pp. 120–124. Flood in Rio Grande Valley in August 1929 affected area from two to five miles wide by about 50 miles long, with population of about 350 families. One water supply was imperiled, from large dug well with solid masonry walls. Flood water came very close to top of curb, but apparently did not enter well. As precautionary measure, well was treated daily with 12 ounces of chloride of lime. All dug wells in area were cleaned where necessary and dosed with chloride of lime. Water from deep drilled wells was available for refugee camps. Refugees were vaccinated against typhoid. **Status of Municipal Sanitary Improvements in Texas.** List as of January 1, 1930, of

water supply and sewerage systems in Texas. Data include name of place, population, depth of wells, ownership of water supply, character of treatment, if any, ownership of sewer system, type of treatment works, and name of receiving stream.—*J. H. O'Neill.*

Yearbook, Pennsylvania Dept. of Health, 1929: Report of Bureau of Engineering. W. L. STEVENSON. Complete lists, with essential data, of Pennsylvania water filtration plants, chlorination plants, and water softening plants as of December 31, 1929. At that time the state had a population estimated at 9,900,000, of which 6,000,000 used water both filtered and chlorinated and 2,000,000 used chlorinated water, not needing filtration at this time, a total of 8,000,000 people or 80 per cent of the citizens of Pennsylvania. Extension of such safe water to greater numbers year by year is coincident with constantly decreasing typhoid fever rate and unquestionably has been important factor in this decline. With regard to stream pollution, which is regulated by Sanitary Water Board, it is Board's method to inaugurate a coöperative program for a comprehensive clean-up of a stream system. One outstanding example of such a clean-up is that of the Beaver and Shenango Rivers in which the sewage from all but two of the 22 sewered communities on these drainage areas is treated prior to its discharge into these rivers.—*G. C. Houser.*

Public Health in Cattaraugus County, N. Y.; Seventh Annual Report, 1929. R. M. ATWATER. At beginning of year, out of total of 20 public water supplies under supervision, 14 at times showed evidence of pollution and might be considered to be of uncertain sanitary quality. Intensive efforts have reduced number of unsatisfactory supplies to 7. The laboratory has studied the significance of salamanders as sources of pollution of water supplies. From these studies it has been shown that the free living salamanders present in springs and other ground water supplies give off many organisms into the water which are indistinguishable by ordinary means from colon bacilli of human origin.—*G. C. Houser.*

Experimental Studies of Natural Purification in Polluted Waters. V. The Selection of Dilution Waters for Use in Oxygen Demand Tests. EMERY J. THERIAULT, PAUL D. McNAMEE, and CHESTER T. BUTTERFIELD. Public Health Reports, 46: 19, 1084–1116, May 8, 1931. Authors conclude that composition and degree of mineralization of such dilution waters need not be critically adjusted, provided that observations are to be restricted to first stage of deoxygenation, or to first 8 or 10 days of incubation, in work with sewage and industrial wastes. In work where maximum of precision is desired, especially for first few days, consideration should be given to character of seeding, as indicated by presence of plankton capable of growing in highly diluted sewage and by a general infection of aërobic bacteria. Ordinary sewage usually fills these conditions. Exceptional precautions should seldom be required in conduct of usual 5-day oxygen demand test. Difficulties of nitrification in dilute sewage mixtures are essentially those inherent in pure culture work. Probably a rough measure of pH control must be envisaged. It has been shown that nitrite formation is dependent on presence of relatively

large amounts of dissolved oxygen.   This peculiarity of nitrifying organisms may account for the 30 to 60 per cent rule given by Royal Commission (British) on Sewage Disposal regarding permissible limits of oxygen depletion.   In 5-day oxygen demand of raw sewage, using ordinary distilled water as diluent, results generally were 10 per cent lower than in comparative tests with mineralized dilution waters.   Over shorter periods of incubation, percentage error may be considerably greater.   In work with sewage effluents at dilutions of 1 to 5, or less, however, enough mineral salts should be added with sample to furnish suitable degree of mineralization.   Favorable results of workers using distilled water in tests of sewage effluents may be due to retardation of nitrification process by carbonaceous matters present in river waters used as controls.   With development of readily prepared synthetic waters, use of distilled water for dilution purposes has become inadvisable.   In studies of first stage of deoxygenation, and with view to eventual development of dilution water for general use in oxygen demand tests, it appears desirable to standardize on readily prepared phosphate dilution water, without addition of other salts. For more restricted use, range of pH 7 to 8, simple bicarbonate solution of MOHLMAN and associates may be fully as serviceable as the somewhat more complex phosphate mixtures.   Further information is needed regarding pH values reached by sewage effluents and, especially, regarding cultural characteristics of nitrifying organisms in exceedingly dilute solutions encountered in sewage treatment.   Tables (21) and references (24) are included.—*R. E. Noble.*

Experimental Studies of Natural Purification in Polluted Waters. VI. Rate of Disappearance of Oxygen in Sludge of Grossly Polluted Water. EMERY J. THERIAULT and PAUL D. McNAMEE.   Public Health Reports, 46: 22, 1301-1319, May 29, 1931.   Experiments were undertaken in January, 1929, to find an explanation of remarkably high rate of purification observed when grossly polluted water was passed through series of shallow troughs about 1 mile long, in which uniform rate of flow could be maintained.   Heavy mat of sludge soon forms on the small stones or pebbles with which bottom is lined, simulating a very small stream, or a trickling filter with horizontal disposition of the filter bed instead of vertical.   All parts are readily accessible for sampling, permitting observations of the sequence of changes which occur in natural purification.   This study deals with the rates of purification between selected points under controlled conditions.   Deoxygenation of polluted water, its mathematical expression, and technical considerations involved are discussed. Experiments under carefully controlled temperature conditions indicated that, for the relatively rapid reaction at least, derivation of a temperature coefficient would be hazardous without taking into consideration pH value of medium and degree of staleness of sludge.   Immediate oxygen demand is responsive to treatments which should tend to increase putrescibility, or degree of staleness, of samples.   Despite the tremendous increase in oxygen requirement of stored samples, it does not appear that the specific rates of deoxygenation were materially affected when the sludge was subjected to storage prior to aëration.   Oxygen demand of sludge under test was not greatly affected by variation of 0.4 pH unit.   Neither did presence of phos-

phate, as such, exert any material effect on rate, or extent, of deoxygenation. Dilution within a fourfold range is without material effect on the specific rate of satisfaction of immediate demand. Latter, although relatively high, does not even approach the observed rate of disappearance of organic matter under channel conditions. In all probability the rapid diminution of organic matters in supernatant liquor is due to rapid adsorption by the sludge and not to oxidation. Relative to rate of adsorption of organic matter by aërated sludge, it is only under special conditions that oxygen-demand results obtained over arbitrarily selected periods of incubation can be applied with any degree of confidence to estimation of percentage removal of organic matter effected by sewage-treatment devices. Correspondence between observed and experimental rates of purification is satisfactory and would not be greatly altered by minor changes in assumed values of deoxygenation constants. Authors draw the general conclusion that, in the sense of actual oxidation, purification by aërated sludge under channel conditions probably does not occur at a rate materially different from that observed under laboratory conditions at same temperature. Experiments suggest the possibility of striking an oxygen balance in small streams where sludge mat is presumably responsible for high degree of apparent biological efficiency. Diagrams of apparatus for circulation of gases in a closed system and for study of air oxidations in liquid media, are shown. Three tables, 10 graphs, and 3 references are included.—*R. E. Noble.*

Detailed Instructions for the Performance of the Dissolved Oxygen and Biochemical Oxygen Demand Tests. EMERY J. THERIAULT. Public Health Reports, Supplement No. **90**: 34 pages. **Part I. Detailed Instructions for the Determination of Dissolved Oxygen by the Winkler Method.** WINKLER process for determination of dissolved oxygen depends on formation of precipitate of manganous hydroxide in glass-stoppered bottle completely filled with water under examination. Oxygen dissolved in the water is rapidly absorbed by the manganous hydroxide, forming higher oxide of uncertain composition which, on acidification in presence of iodide, releases iodine in quantity stoichiometrically equivalent to oxygen content of sample. Liberated iodine is then titrated with standard solution of sodium thiosulphate. Preparation and properties of 16 reagents are given. Apparatus consists of (1) sample bottles of 240–380 cc. capacity, (2) sampling devices for simultaneous collection of bacteriological, oxygen demand, and dissolved oxygen samples, (3) special pipettes, and (4) dosing stand. Sampling arrangements should be such that liquid in sampling bottle is displaced several times without entrainment of air bubbles. The selection of a method of procedure is important since presence of nitrites, ferrous salts, organic matter, sulphites, etc., or improper application of modifications designed to overcome them, may introduce huge errors in dissolved oxygen or oxygen demand work. Nitrites will cause interference when present in amounts exceeding 0.1 p.p.m. of N as NO_2. Absent at the start, nitrites may appear during incubation. 1 p.p.m. of ferrous iron (Fe) will occasion an apparent loss of about 0.14 p.p.m. of dissolved oxygen. The RIDEAL-STEWART (permanganate) modification of WINKLER method cannot be recommended as corrective for interference due to rela-

tively stable forms of organic matter such as sugars, starches, etc. It also fails in presence of sulphite wastes. Detailed instructions are given for the determination of dissolved oxygen by (A) Permanganate (RIDEAL-STEWART) modification, suitable in presence of nitrites, ferrous salts, etc., (B) Alkaline-hypochlorite modification, applicable to pure waters and also to liquors heavily polluted with relatively stable forms of organic matter in absence of readily oxidizable mineral or organic substances, and (C) Original WINKLER method, suitable in presence of sulphite-waste liquor (sulphites, thiosulphates, polythionates, etc.). These have been condensed as follows: *Preliminary Treatment with Permanganate and Fluoride.* (1) Add 0.7 cc. concentrated sulphuric acid. (2) Add enough permanganate solution to obtain a violet tinge. Add 2 cc. of fluoride solution if appreciable amounts of iron salts are present. (3) Allow to stand until reaction of permanganate with readily oxidizable substances is judged complete (5 minutes or less). (4) Decolorize with the minimum possible amount of oxalate solution. The decolorization should proceed in the dark when iron salts are present. *Preliminary Treatment with Alkaline-Hypochlorite.* (1) Add suitable amount of alkaline-hypochlorite reagent. (2) Shake for 20 seconds. (3) Add 1 cc. each of 3.5 N H_2SO_4 (10 per cent solution) and 1 M KI. Shake. (4) Add 0.2 cc. of starch solution and neutralize the iodine with 0.05 M (0.1 N) sulphite solution, restoring the blue color with 0.1 cc. portions of 0.1 N biniodate. *The Winkler Procedure.* (5) After preliminary treatment with permanganate, add 1 cc. manganous sulphate and 3 cc. alkaline iodide: after preliminary treatment with alkaline hypochlorite, add 1 cc. manganous sulphate and 1.3 cc. alkaline iodide: when preliminary treatment unnecessary, add 2 cc. each of manganous sulphate and alkaline iodide. Lesser amounts may be added under certain conditions. (6) In practical absence of organic matter it is permissible to shake each bottle in turn vigorously for a few seconds; a second shaking is advisable. In presence of much organic matter treat each sample individually as directed in detailed instructions. (7) Acidify with concentrated sulphuric acid; 1, or 2 cc., depending on conditions. (8) Distribute liberated iodine uniformly throughout bottle. *The Titration.* (9) In general, titrations should be commenced within one hour after final acidification. (Tolerance in special cases.) (10) For titration it is convenient to pour out a portion of the iodine solution which corresponds to 100 or 200 cc. of original sample. (11) Titrate to pale straw color with 0.025 M thiosulphate. (12) Add 1 or 2 cc. starch solution and titrate rapidly to first disappearance of blue color, neglecting subsequent recolorations. (13) Titration should not be delayed, once sample has been poured out. *Computation.* (14) In general,

$$\text{p.p.m. of dissolved oxygen} = \frac{200 \ (\text{cc. } 0.025 \ M \ \text{thiosulphate})}{(\text{cc. of sample titrated})}.$$

Part II. Detailed Instructions for the Determination of Bio-Chemical Oxygen Demand by the Excess-Oxygen (Dilution) Method. Method consists, essentially, in measuring decrease in dissolved oxygen which occurs when liquid containing organic matter and capable of supporting biological growth is incubated under standard conditions. As water at ordinary temperatures will hold only about 9 p.p.m. of dissolved oxygen in equilibrium with air, it is necessary to dilute the samples to secure an excess of dissolved oxygen when-

ever oxygen demand exceeds 7 or 8 p.p.m.   Other complications are introduced when samples have to be seeded, neutralized, etc., prior to incubation.   With suitable working arrangements, test can be carried out rapidly and conveniently.   Close adherence to analytical details is necessary to obtain consistent results.   Apparatus consists of (1) sampling devices described in Part I, (2) low form incubation bottles provided with accurately fitting glass stoppers: a 300-cc. bottle of preferred type is described, (3) 20-liter carboys, (4) seals, (5) special siphons and pipettes, and (6) 20°C. incubator, capable of regulation within 1°.   In the ideal case, hourly samples, proportioned to flow of waste at time of sampling (e.g. 10 cc. per m.g., if average flow is 10 million gallons) are collected over a full 24-hour period and composited for analysis.   In absence of flow-measuring devices, hourly samples of 150 cc. may be collected throughout the day for compositing.   Dilution water should possess following characteristics at the time of use: (a) 10-day oxygen demand should be less than 0.6 p.p.m., or 5-day demand should be less than 0.4 p.p.m.; (b) it should neither be supersaturated, nor greatly undersaturated, with dissolved oxygen at standard temperature of 20°C.; but should contain between 8 and 9 p.p.m. dissolved oxygen; (c) temperature should be approximately 20°C.; (d) it should be free from interfering substances, such as iron salts and nitrites, and from inhibitory substances, such as free chlorine, chloramines, and copper salts; (e) pH value and mineral salt content should be favorable to biological growth: with development of readily prepared synthetic waters, use of distilled water for dilution purposes has become inadvisable.   In preliminary treatment of sample, consideration must be given to (1) adjustment of pH value, (2) seeding, (3) supersaturation, and (4) undersaturation.   Biological activity may be inhibited if sample is unduly acid (pH 5.5, or less), or unduly alkaline (pH 8.5, or greater): adjustment to a standard value (pH 7.2), or else to that of receiving body of water (pH 6.5 to 8.0) is advisable.   Seeding may be accomplished by adding stale sewage (1 cc., or less, per liter), or river water (10 to 20 cc. per liter) to dilution water.   Concerning a choice of concentration, dilution is seldom necessary in the case of ordinary river water.   For average American sewage, trial concentrations of 2.5 to 5 per cent are suggested.   Starting with a waste which must be diluted prior to incubation, essential requirement in preparation of a series of concentrations is that subsamples be uniform in regard both to distribution of sample and to dissolved oxygen content.   Initial dissolved oxygen content of dilution water must be determined in all cases.   If oxygen content of waste is also known, initial oxygen content of diluted waste may readily be computed.   If dilution technique is such that oxygen content of dilution water is not greatly changed during manipulation, oxygen content of undiluted waste may be approximated from value found for oxygen content of diluted mixture.   Three cases are given.   Fraction of total oxygen demand which is satisfied during first few minutes after dilution is generally referred to as the "immediate oxygen demand."   Under favorable conditions, seals do not appear essential for periods of incubation of about five days at 20°C.   Under less favorable conditions, volume changes may be provided for by either (a) "jointing" the sample with a rubber collar, (b) using bottle provided with a large cup, or (c) immersing samples in water.   The 5-day demand appears

to bear the most definite relation to the values obtained over other periods of incubation. At 20°C. a variation in temperature of one degree causes a change of about 5 per cent in specific rate of deoxygenation. Temperature should be kept reasonably constant. A correction must be applied if dilution water itself has an appreciable oxygen demand, hence dilution water blanks are incubated with diluted subsamples. Correction should be proportioned to amount of dilution water present. *Computation.* Given initial and final oxygen-contents of a diluted sample over any period of time, oxygen demand of the sample for that period is the corrected difference times a factor for the concentration used. The concentration factor is the reciprocal of the concentration when expressed decimally. Thus for a 5 per cent concentration, or decimally 0.05, the factor is 20.—*R. E. Noble.*